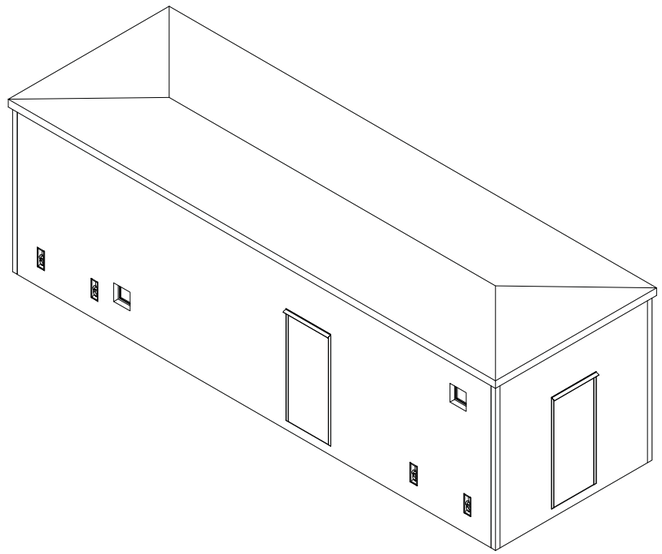
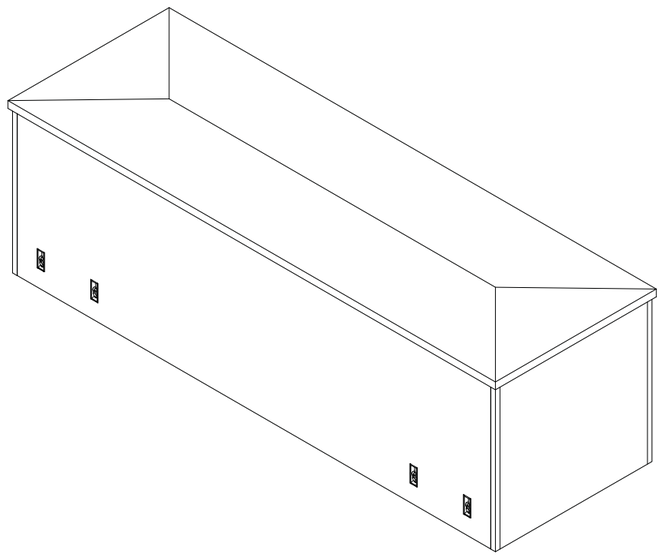


1 3D VIEW 1



2 3D VIEW 2

DRAWING INDEX

SHEET #	SHEET DESCRIPTION	REV. #	REV. DATE	REV. DESCRIPTION
G0.0	COVER PAGE			
G0.1	PROJECT DATA FOR 30'-36' LENGTHS			
G0.2	PROJECT DATA FOR 12'-28' LENGTHS			
G0.3	GENERAL SPECIFICATIONS			
G0.4	ABBREVIATIONS AND SYMBOLS			
A1.1	FLOOR PLAN			
A1.2	ROOF PLAN			
A1.3	EXTERIOR ELEVATIONS - PANEL A & B			
A1.4	EXTERIOR ELEVATIONS - PANEL C & D			
A2.0A	INTERIOR FINISH & MISC. DETAILS - OPTION A			
A2.0B	INTERIOR FINISH & MISC. DETAILS - OPTION B			
A2.0C	INTERIOR FINISH & MISC. DETAILS - OPTION C			
S0.1	OPENING REINFORCEMENT DETAILS AND LEGEND			
S1.0	FOUNDATION NOTES & INFORMATION			
S1.1A	PERIMETER BEAM FOUNDATION PLAN & DETAILS			
S1.1B	SLAB ON GRADE FOUNDATION PLAN & DETAILS			
S1.6	FOUNDATION - BEARING PAD & SLIDE PLATE PLAN			
S1.7	TYPICAL FOUNDATION DETAILS			
S2.1	8" FLOOR PANEL			
S2.2	8" FLOOR PANEL - SECTIONS			
S2.3	8" FLOOR PANEL - REBAR CAGE & JOIST DETAILS			
S3.1	6", 5", & 4" ROOF PANEL(S)			
S4.1	4" ENDWALL PANEL A			
S4.2	4" SIDEWALL PANEL B			
S4.3	4" ENDWALL PANEL C			
S4.4	4" SIDEWALL PANEL D			
S7.0	BUILDING SECTIONS			
S8.0	8" FLOOR CONNECTION DETAILS			
S8.1	6", 5", & 4" ROOF CONNECTION DETAILS			
S8.2	4" WALL TO WALL CONNECTION & MISC. DETAILS			
S9.0	MISCELLANEOUS DETAILS			
S10.1	CENTER-OF-GRAVITY PLAN (SIDEWALL LIFT - 8)			
S10.2	CENTER-OF-GRAVITY PLAN (SIDEWALL LIFT - 4)			
Grand total: 33				

MODULAR CONNECTIONS

MODULAR CONNECTIONS, LLC

CONCRETE BUILDING

1090 INDUSTRIAL BLVD.
BESSEMER, AL 35022

PHONE: 205-980-4565
FAX: 877-675-5851

Columbia County Building Department
Plans Reviewed for Code Compliance
State of Florida

DATE 5/30/2024 CERT. NO SMP-056
PLAN NUMBER MFT4193-Standard Concret
APPROVED BY MIchael Faller

(signature)

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

NOTES:

1. LIFTING BRACKETS ARE RATED FOR THE POSTED LOADS ON THE S4 AND S10 SHEETS. IT IS THE RESPONSIBILITY OF THE TRANSPORTATION CONTRACTOR TO ENSURE INSTALLED EQUIPMENT, PACK-IN-SHELTER ITEMS ARE ARRANGED IN A MANNER SUCH THAT EACH LIFTING BRACKET'S CAPACITY IS NOT EXCEEDED.

STATE: **FLORIDA**

LONNIE A MARVEL LICENSE No. 83750 STATE OF FLORIDA PROFESSIONAL ENGINEER

2024.05.29 15:26:15-04'00'

No. | Revision Description | Date | By

MODULAR CONNECTIONS
1090 Industrial Blvd.
Bessemer, AL 35022
PH# 205-980-4565
Email: info@modularconnections.com

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TP ENGINEERING, INC.
Design & Inspection Services
www.tpengineeringinc.com
205-655-9992

Title: 12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site: IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name: COVER PAGE

Project Number: Sheet Number: G0.0

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

NOTICE: These drawings and specifications are the property of Modular Connections, LLC. All information contained herein which is not known generally in the field of Modular Connections, LLC shall be confidential except to any extent to which it is established to have been known previously from sources other than Modular Connections, LLC. These drawings and specifications may not be reproduced, copied or used as the basis for the manufacture or sale of apparatus without written permission.

PROJECT DATA - FLORIDA

DESIGN CODES:

8TH EDITION (2023) FLORIDA BUILDING CODE
8TH EDITION (2023) FLORIDA ENERGY CONSERVATION CODE
8TH EDITION (2023) FLORIDA FIRE PREVENTION CODE
8TH EDITION (2023) FLORIDA FUEL GAS CODE
2020 NATIONAL ELECTRIC CODE (NFPA 70)
8TH EDITION (2023) FLORIDA PLUMBING CODE
8TH EDITION (2023) FLORIDA MECHANICAL CODE
8TH EDITION (2023) FLORIDA BUILDING CODE: ACCESSIBILITY

① DESIGN CODES
12" = 1'-0"

SIZE SCHEDULE

(SEE SCHEDULE NOTES BELOW - THIS SHEET)

PART NO.	(W) EXTERIOR	(L) EXTERIOR	(H) INTERIOR
ESC1230	12'-0"	30'-0"	8'-0"
ESC1232	12'-0"	32'-0"	8'-0"
ESC1234	12'-0"	34'-0"	8'-0"
ESC1236	12'-0"	36'-0"	8'-0"
ESC12309	12'-0"	30'-0"	9'-0"
ESC12329	12'-0"	32'-0"	9'-0"
ESC12349	12'-0"	34'-0"	9'-0"
ESC12369	12'-0"	36'-0"	9'-0"
ESC123010	12'-0"	30'-0"	9'-8"
ESC123210	12'-0"	32'-0"	9'-8"
ESC123410	12'-0"	34'-0"	9'-8"
ESC123610	12'-0"	36'-0"	9'-8"

SCHEDULE NOTES:

1. BUILDING WIDTH (W) INDICATED IS TO OUTSIDE EDGE OF ROOF.
2. BUILDING LENGTH (L) INDICATED IS TO OUTSIDE EDGE OF ENDWALLS.
3. BUILDING HEIGHT (H) INDICATED IS NOMINAL INTERIOR CLEAR HEIGHT.

DESIGN LOADS: 30' - 36' LENGTHS (SEE SIZE SCHEDULE THIS SHEET)

FLOOR LIVE LOAD	FLOOR OPTION #1 = **250 PSF**, FLOOR OPTION #2 = **200 PSF**, FLOOR OPTION #3 = **150 PSF**
FLOOR DEAD LOAD	37.3 - 37.6 PSF, VARIES WITH BUILDING LENGTH
RISK CATEGORY	II
GROUND SNOW LOAD	31.5 PSF
SNOW LOAD IMPORTANCE FACTOR	1
SNOW EXPOSURE FACTOR	1.2
THERMAL FACTOR	1.2
FLAT ROOF SNOW LOAD	31.8 PSF
WIND VELOCITY (3-SEC GUST, Vult)	150 MPH
WIND VELOCITY (3-SEC GUST, Vasd)	116 MPH
WIND EXPOSURE CATEGORY	C
INTERNAL PRESSURE COEFFICIENT	±0.18
WIND LOAD HORIZONTAL (MWFRS)	35.8 PSF
WIND LOAD (UPLIFT) (MWFRS)	53.5 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 4)	53.3 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 5)	65.8 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 1)	78.2 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 2)	103.2 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 3)	140.7 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 4, Vasd)	31.9 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 5, Vasd)	39.4 PSF
SEISMIC IMPORTANCE FACTOR	1
SPECTRAL RESPONSE ACCELERATION S_{D1}	0.839
SEISMIC DESIGN CATEGORY	D
SITE CLASS	D
BASIC SEISMIC-FORCE RESISTING SYSTEM	A.5 - BEARING WALL SYSTEMS - INTERMEDIATE PRECAST SHEAR WALLS
ANALYSIS PROCEDURE	EQUIVALENT LATERAL-FORCE PROCEDURE
OCCUPANCY USE GROUP	S2
CONSTRUCTION TYPE	VB
MINIMUM SETBACK	SEE NOTES FOR A COMMON OR ASSUMED PROPERTY LINE TO COMPLY WITH IBC-2021

ROOF OPTION #1 100 PSF:

ROOF LIVE LOAD	ROOF OPTION #1 = 100 PSF
ROOF DEAD LOAD	ROOF OPTION #1 = 66.4 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #1 = 0.579
DESIGN BASE SHEAR	ROOF OPTION #1 = 0.145W

ROOF OPTION #2 80 PSF:

ROOF LIVE LOAD	ROOF OPTION #2 = 80 PSF
ROOF DEAD LOAD	ROOF OPTION #2 = 56.5 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #2 = 0.629
DESIGN BASE SHEAR	ROOF OPTION #2 = 0.157W

ROOF OPTION #3 35 PSF:

ROOF LIVE LOAD	ROOF OPTION #3 = 35 PSF
ROOF DEAD LOAD	ROOF OPTION #3 = 46.7 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #3 = 0.777
DESIGN BASE SHEAR	ROOF OPTION #3 = 0.194W

② DESIGN LOADS FOR 30' - 36' LENGTHS
12" = 1'-0"

NOTES:

1. THE INTENDED OCCUPANCY OF THIS EQUIPMENT BUILDING IS FOR PERFORMING THE REQUIRED SERVICE/MAINTENANCE OF EQUIPMENT ONLY.
2. PER IBC, 4" SAND-LIGHTWEIGHT CONCRETE WALLS HAVE FIRE RESISTANCE RATING OF 2 HOURS WITH UNPROTECTED OPENINGS.
3. ALL SETBACK REQUIREMENTS SHALL CONFORM TO THE REFERENCED IBC AND LOCAL CODE REQUIREMENTS.
4. THIS BUILDING IS REQUIRED TO BE CONNECTED TO PUBLIC UTILITIES.
5. THE USE OF THIS BUILDING WITHOUT RESTROOM FACILITIES IS SUBJECT TO LOCAL REVIEW AND APPROVAL.
6. THESE PLANS COMPLY WITH RULE 61G20-3.006 (PRODUCT APPROVAL)
7. THE SEALED SET OF PLANS ARE ON FILE IN THE THIRD AGENCY'S OFFICE AS DIRECTED BY DBPR.
8. THIS BUILDING IS NOT APPROVED FOR CHAPTER 633 FIRE SAFETY CODE AND IS SUBJECT TO REVIEW AND INSPECTION BY FIRE INSPECTOR ON SITE.
9. BUILDINGS IN COASTAL AREAS THAT ARE SUBJECT TO SALT WATER SPRAY SHALL HAVE THE CONCRETE AT A MINIMUM OF 5000PSI @ 28 DAYS.
10. PLACEMENT OF THE PROJECT DATA PLATE AND THE DBPR INSIGNIA MUST BE ON OR NEAR THE MAIN ELECTRICAL PANEL.
11. THESE STRUCTURES ARE NOT DESIGNED OR APPROVED FOR PLACEMENT IN HIGH-VELOCITY HURRICANE ZONES (HVHZ), IE. DADE OR BROWARD COUNTIES.
12. THESE STRUCTURES HAVE BEEN DESIGNED FOR ERECTION OR INSTALLATION ON A SITE-BUILT PERMANENT FOUNDATION AND IS NOT DESIGNED TO BE MOVED ONCE SO ERECTED OR INSTALLED.
13. MODULAR CONNECTIONS FLORIDA CERTIFICATION ID MFT-4193.
14. THESE STRUCTURES ARE NOT DESIGNED FOR FLOOD HAZARD AREAS.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

STATE: **FLORIDA**

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
2024.05.29 15:26:21-04'00'

No	Revision Description	Date	By

MODULAR CONNECTIONS
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565
Email: info@modularconnections.com

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TP ENGINEERING, INC.
Design & Inspection Services
www.tpengineeringinc.com
205-655-9992

Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
PROJECT DATA FOR 30'-36' LENGTHS

Project Number: Sheet Number G0.1

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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PROJECT DATA - FLORIDA

DESIGN CODES:

8TH EDITION (2023) FLORIDA BUILDING CODE
8TH EDITION (2023) FLORIDA ENERGY CONSERVATION CODE
8TH EDITION (2023) FLORIDA FIRE PREVENTION CODE
8TH EDITION (2023) FLORIDA FUEL GAS CODE
2020 NATIONAL ELECTRIC CODE (NFPA 70)
8TH EDITION (2023) FLORIDA PLUMBING CODE
8TH EDITION (2023) FLORIDA MECHANICAL CODE
8TH EDITION (2023) FLORIDA BUILDING CODE: ACCESSIBILITY

① DESIGN CODES
12" = 1'-0"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SIZE SCHEDULE

(SEE SCHEDULE NOTES BELOW - THIS SHEET)

PART NO.	(W) EXTERIOR	(L) EXTERIOR	(H) INTERIOR
ESC1212	12'-0"	12'-0"	8'-0"
ESC1214	12'-0"	14'-0"	8'-0"
ESC1216	12'-0"	16'-0"	8'-0"
ESC1218	12'-0"	18'-0"	8'-0"
ESC1220	12'-0"	20'-0"	8'-0"
ESC1222	12'-0"	22'-0"	8'-0"
ESC1224	12'-0"	24'-0"	8'-0"
ESC1226	12'-0"	26'-0"	8'-0"
ESC1228	12'-0"	28'-0"	8'-0"
ESC12129	12'-0"	12'-0"	9'-0"
ESC12149	12'-0"	14'-0"	9'-0"
ESC12169	12'-0"	16'-0"	9'-0"
ESC12189	12'-0"	18'-0"	9'-0"
ESC12209	12'-0"	20'-0"	9'-0"
ESC12229	12'-0"	22'-0"	9'-0"
ESC12249	12'-0"	24'-0"	9'-0"
ESC12269	12'-0"	26'-0"	9'-0"
ESC12289	12'-0"	28'-0"	9'-0"
ESC121210	12'-0"	12'-0"	9'-8"
ESC121410	12'-0"	14'-0"	9'-8"
ESC121610	12'-0"	16'-0"	9'-8"
ESC121810	12'-0"	18'-0"	9'-8"
ESC122010	12'-0"	20'-0"	9'-8"
ESC122210	12'-0"	22'-0"	9'-8"
ESC122410	12'-0"	24'-0"	9'-8"
ESC122610	12'-0"	26'-0"	9'-8"
ESC122810	12'-0"	28'-0"	9'-8"

SCHEDULE NOTES:

1. BUILDING WIDTH (W) INDICATED IS TO OUTSIDE EDGE OF ROOF.
2. BUILDING LENGTH (L) INDICATED IS TO OUTSIDE EDGE OF ENDWALLS.
3. BUILDING HEIGHT (H) INDICATED IS NOMINAL INTERIOR CLEAR HEIGHT.

DESIGN LOADS: 12' - 28' LENGTHS (SEE SIZE SCHEDULE THIS SHEET)

FLOOR LIVE LOAD	FLOOR OPTION #1 = **250 PSF**, FLOOR OPTION #2 = **200 PSF**, FLOOR OPTION #3 = **150 PSF**
FLOOR DEAD LOAD	37.3 - 37.6 PSF, VARIES WITH BUILDING LENGTH
RISK CATEGORY	II
GROUND SNOW LOAD	31.5 PSF
SNOW LOAD IMPORTANCE FACTOR	1
SNOW EXPOSURE FACTOR	1.2
THERMAL FACTOR	1.2
FLAT ROOF SNOW LOAD	31.8 PSF
WIND VELOCITY (3-SEC GUST, Vult)	165 MPH
WIND VELOCITY (3-SEC GUST, Vasd)	127 MPH
WIND EXPOSURE CATEGORY	C
INTERNAL PRESSURE COEFFICIENT	±0.18
WIND LOAD HORIZONTAL (MWFRS)	43.1 PSF
WIND LOAD (UPLIFT) (MWFRS)	64.7 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 4)	64.5 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 5)	79.6 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 1)	94.7 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 2)	124.9 PSF
WIND LOAD (UPLIFT) (C & C, ZONE 3)	170.2 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 4, Vasd)	38.7 PSF
WIND LOAD HORIZONTAL (C & C, ZONE 5, Vasd)	47.6 PSF
SEISMIC IMPORTANCE FACTOR	1
SPECTRAL RESPONSE ACCELERATION S_{D1}	0.839
SEISMIC DESIGN CATEGORY	D
SITE CLASS	D
BASIC SEISMIC-FORCE RESISTING SYSTEM	A.5 - BEARING WALL SYSTEMS - INTERMEDIATE PRECAST SHEAR WALLS
ANALYSIS PROCEDURE	EQUIVALENT LATERAL-FORCE PROCEDURE
OCCUPANCY USE GROUP	S2
CONSTRUCTION TYPE	VB
MINIMUM SETBACK	SEE NOTES FOR A COMMON OR ASSUMED PROPERTY LINE TO COMPLY WITH IBC-2021

ROOF OPTION #1 100 PSF:

ROOF LIVE LOAD	ROOF OPTION #1 = 100 PSF
ROOF DEAD LOAD	ROOF OPTION #1 = 66.4 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #1 = 0.773
DESIGN BASE SHEAR	ROOF OPTION #1 = 0.183W

ROOF OPTION #2 80 PSF:

ROOF LIVE LOAD	ROOF OPTION #2 = 80 PSF
ROOF DEAD LOAD	ROOF OPTION #2 = 56.5 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #2 = 0.801
DESIGN BASE SHEAR	ROOF OPTION #2 = 0.2W

ROOF OPTION #3 35 PSF:

ROOF LIVE LOAD	ROOF OPTION #3 = 35 PSF
ROOF DEAD LOAD	ROOF OPTION #3 = 46.7 PSF
SPECTRAL RESPONSE ACCELERATION S_{DS}	ROOF OPTION #3 = 0.96
DESIGN BASE SHEAR	ROOF OPTION #3 = 0.24W

② DESIGN LOADS FOR 12' - 28' LENGTHS
12" = 1'-0"

STATE: **FLORIDA**

NOTES:

1. THE INTENDED OCCUPANCY OF THIS EQUIPMENT BUILDING IS FOR PERFORMING THE REQUIRED SERVICE/MAINTENANCE OF EQUIPMENT ONLY.
2. PER IBC, 4" SAND-LIGHTWEIGHT CONCRETE WALLS HAVE FIRE RESISTANCE RATING OF 2 HOURS WITH UNPROTECTED OPENINGS.
3. ALL SETBACK REQUIREMENTS SHALL CONFORM TO THE REFERENCED IBC AND LOCAL CODE REQUIREMENTS.
4. THIS BUILDING IS REQUIRED TO BE CONNECTED TO PUBLIC UTILITIES.
5. THE USE OF THIS BUILDING WITHOUT RESTROOM FACILITIES IS SUBJECT TO LOCAL REVIEW AND APPROVAL.
6. THESE PLANS COMPLY WITH RULE 61G20-3.006 (PRODUCT APPROVAL)
7. THE SEALED SET OF PLANS ARE ON FILE IN THE THIRD AGENCY'S OFFICE AS DIRECTED BY DBPR.
8. THIS BUILDING IS NOT APPROVED FOR CHAPTER 633 FIRE SAFETY CODE AND IS SUBJECT TO REVIEW AND INSPECTION BY FIRE INSPECTOR ON SITE.
9. BUILDINGS IN COASTAL AREAS THAT ARE SUBJECT TO SALT WATER SPRAY SHALL HAVE THE CONCRETE AT A MINIMUM OF 5000PSI @ 28 DAYS.
10. PLACEMENT OF THE PROJECT DATA PLATE AND THE DBPR INSIGNIA MUST BE ON OR NEAR THE MAIN ELECTRICAL PANEL.
11. THESE STRUCTURES ARE NOT DESIGNED OR APPROVED FOR PLACEMENT IN HIGH-VELOCITY HURRICANE ZONES (HVHZ), IE. DADE OR BROWARD COUNTIES.
12. THESE STRUCTURES HAVE BEEN DESIGNED FOR ERECTION OR INSTALLATION ON A SITE-BUILT PERMANENT FOUNDATION AND IS NOT DESIGNED TO BE MOVED ONCE SO ERECTED OR INSTALLED.
13. MODULAR CONNECTIONS FLORIDA CERTIFICATION ID MFT-4193.
14. THESE STRUCTURES ARE NOT DESIGNED FOR FLOOD HAZARD AREAS.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER
2024.05.29 15:26:26-04'00'

No	Revision Description	Date	By

MODULAR CONNECTIONS

1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565
Email: info@modularconnections.com

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TP ENGINEERING, INC.
Design & Inspection Services
www.tpengineeringinc.com
205-655-9992

Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
PROJECT DATA FOR 12'-28' LENGTHS

Project Number:

Sheet Number
G0.2

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one quarter inch = one foot
0 4 8

three eighths inch = one foot
0 4

one half inch = one foot
0 4

three quarters inch = one foot
0 2

one and one half inch = one foot
0 1

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GENERAL SPECIFICATIONS

1. **SCOPE**
   - A. THE SPECIFICATIONS CONTAINED HEREIN ENCOMPASS THE LABOR, EQUIPMENT AND MATERIALS FOR THE FABRICATION OF A TRANSPORTABLE, BULLET RESISTANT, PRE-CAST CONCRETE STRUCTURE.
     - a. THE BUILDING SHALL WITHSTAND 30/06 RIFLE FIRE AT 15' DISTANCE PER UL752 STANDARDS.

2. **GENERAL**
   - A. BUILDING TYPE:
     - a. THE BUILDING SHALL BE PRECAST, PRE-ASSEMBLED CONCRETE.
   - B. BUILDING SIZES:
     - a. 12' WIDE BY 30'-36' LENGTH IN 2'-0" INCREMENTS.
   - C. OPERATING ENVIRONMENT:
     - a. THE BUILDING SHALL BE SEALED TO RESIST DUST INFILTRATION AND BE WATERTIGHT.
   - D. DESIGN LOADING:
     - a. SEE SHEET G0.1.
   - E. LOCATE STATE DECAL AT LOAD DISTRIBUTION PANEL.

3. **SEALING**
   - A. JOINTS:
     - a. ALL JOINTS SHALL BE SEALED WITH A COMPRESSIBLE, RESILIENT SEALANT.
     - b. THERE SHALL BE NO EXPOSED ROOF TO WALL, WALL TO FLOOR, OR WALL TO WALL JOINTS.
   - B. EXTERIOR WALLS AND ROOF:
     - a. EXTERIOR SURFACES OF WALLS AND ROOF SHALL BE SEALED WITH TWO COATS OF THOROGLAZE H SEALER, OR ACCEPTABLE EQUAL IN STRICT CONFORMANCE WITH MANUFACTURER'S INSTRUCTIONS.

4. **THERMAL**
   - A. THERMAL INSULATION REQUIREMENTS SHALL MEET THE MINIMUM REQUIREMENT PER DESIGN CODES ON SHEET G0.1.
   - B. ADDITIONAL REQUIREMENT REFER TO INTERIOR FINISH SHEETS.

5. **DOOR**
   - A. GENERAL:
     - a. DOOR OPENING(S): VARIES.
   - B. FRAME:
     - a. SHALL BE MINIMUM 1 3/4" 18 GAUGE GALVANIZED STEEL, PRIMED, PAINTED, AND CAST INTO THE WALL PANEL.
   - C. DOOR:
     - a. SHALL BE MINIMUM 1 3/4" 18 GAUGE GALVANIZED STEEL, INSULATED, PRIMED, PAINTED, AND INSTALLED FLUSH.
   - D. HARDWARE:
     - a. INSTALLED WITH DOOR CHECK, DOOR STOP, WEATHER STRIPPING, MORTISE LOCKSET, AND STAINLESS STEEL BALL BEARING HINGES.
   - E. ADDITIONAL REQUIREMENTS REFER TO WALL PANEL SHEETS.
   - F. ADDITIONAL OPTIONS PER CLIENT SPECIFICATIONS.

6. **INTERIOR FINISHES:**
   - A. SEE INTERIOR FINISH SHEETS.

7. **STRUCTURAL**
   - A. STRUCTURAL DESIGN AND MANUFACTURING SHALL CONFORM TO REQUIREMENTS OF ACI 318.
   - B. BUILDING CONCRETE SPECIFICATIONS:
     - a. 28 DAY COMPRESSIVE STRENGTH SHALL BE 5000 PSI.
     - b. MIX DESIGN SHALL BE 118 LB./CU.FT. STRUCTURAL LIGHTWEIGHT CONCRETE USING EXPANDED SHALE OR EXPANDED CLAY AGGREGATE. MIX SHALL BE HOMOGENOUS. SEEDING OF AGGREGATES FOR EXPOSED AGGREGATE FINISH IS NOT ALLOWED.
     - c. MAXIMUM SLUMP SHALL BE 6".
   - C. MATERIALS SPECIFICATIONS:
     - a. CEMENT USED IN CONCRETE SHALL BE STANDARD PORTLAND CEMENT CONFORMING TO THE REQUIREMENTS OF THE "STANDARD SPECIFICATIONS FOR PORTLAND CEMENT," ASTM DESIGNATION C150.
     - b. CONCRETE AGGREGATES SHALL CONFORM TO ONE OF THE FOLLOWING SPECIFICATIONS:
       1. "SPECIFICATIONS FOR CONCRETE AGGREGATES," ASTM DESIGNATION: C33.
       2. "SPECIFICATIONS FOR LIGHTWEIGHT AGGREGATES FOR STRUCTURAL CONCRETE," ASTM DESIGNATION C330.
       3. WATER SHALL BE FREE FROM INJURIOUS QUANTITIES OF OIL, ALKALI, VEGETABLE MATTER AND SALT. NON-POTABLE WATER SHALL NOT BE USED IN MIXING CONCRETE.
       4. REINFORCEMENT BARS SHALL BE DEFORMED STEEL BARS CONFORMING TO THE REQUIREMENTS OF THE "SPECIFICATIONS FOR DEFORMED AND PLAIN BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT," ASTM DESIGNATION: A615. (FY = 60 KSI)
       5. ADMIXTURES SHALL BE SUBJECT TO PRIOR APPROVAL BY THE QUALITY CONTROL SUPERVISOR AND SHALL MAINTAIN THE SAME COMPOSITION AND PERFORMANCE INTENDED IN THE ORIGINAL MIX DESIGN.
       6. AIR-ENTRAINING ADMIXTURES SHALL CONFORM TO "SPECIFICATIONS FOR AIR-ENTRAINING ADMIXTURES FOR CONCRETE," ASTM DESIGNATION: C260.
       7. WATER REDUCING ADMIXTURES, RETARDING ADMIXTURES, ACCELERATING ADMIXTURES, WATER REDUCING AND RETARDING ADMIXTURES, AND WATER REDUCING AND ACCELERATING ADMIXTURES SHALL CONFORM TO "SPECIFICATIONS FOR CHEMICAL ADMIXTURES FOR CONCRETE," ASTM DESIGNATION: C494.
     - c. "SPECIFICATIONS FOR STRUCTURAL STEEL" ASTM DESIGNATION A36.
     - d. ALL WELDING AND WELDED STUDS SHALL CONFORM TO THE REQUIREMENTS OF THE "STRUCTURAL WELDING CODE-STEEL" ANSI/AWS D1.1.
       1. ALL WELDING ELECTRODES SHALL BE E70XX. FOR SHIELDED METAL ARC WELDING (SMAW) USE LOW HYDROGEN ELECTRODES.
       2. ALL WELDING WIRE SHALL BE 0.035 MIG WIRE FOR GAS METAL ARC WELDING (GMAW).
       3. WELDED SMOOTH WIRE FABRIC SHALL BE STEEL WIRE FABRIC CONFORMING TO THE REQUIREMENTS OF THE "SPECIFICATIONS FOR WELDED STEEL WIRE FABRIC FOR CONCRETE REINFORCEMENT," ASTM DESIGNATION: A185.
   - D. FLOOR PANEL:
     - a. SHALL BE AN 8" WAFFLED STRUCTURAL PRECAST CONCRETE SYSTEM.
       1. RIBS SHALL BE 2'-0"O.C. MAXIMUM TRANSVERSE.
       2. FLOOR SLAB SHALL BE 2".
     - b. ALL SURFACES SHALL BE SMOOTH AND PREPPED FOR FINISH(S).
     - c. SEE FLOOR PANEL SHEETS FOR ADDITIONAL REQUIREMENTS.
   - E. ROOF PANEL:
     - a. SHALL BE STRUCTURAL PRECAST CONCRETE WITH A MINIMUM EDGE THICKNESS OF 4", 5", OR 6" ACCORDING TO ROOF LIVE LOAD OPTIONS.
     - b. ROOF SHALL BE HIP TYPE SLOPING 1/4" PER FOOT IN FOUR DIRECTIONS.
     - c. SHALL BE PROVIDE WITH A 3/4" RECESSED KEYWAY FOR TOP OF WALL PANEL LEAVING NO EXPOSED ROOF TO WALL JOINT.
     - d. SHALL PROVIDE A 2" MAXIMUM OVERHANG ON ALL SIDES UNLESS NOTED OTHERWISE.
     - e. SEE ROOF PANEL SHEETS FOR ADDITIONAL REQUIREMENTS.
   - F. WALL PANELS:
     - a. SHALL BE 4" THICK STRUCTURAL PRECAST CONCRETE.
     - b. EXTERIOR FINISH SHALL BE EXPOSED AGGREGATE UNLESS OTHERWISE NOTED.
     - c. INTERIOR SURFACES SHALL BE SMOOTH AND PREPPED FOR FINISH(S).
     - d. WALL PANELS SHALL OVERHANG THE TOP FLOOR A MINIMUM OF 7" LEAVING NO EXPOSED WALL TO FLOOR JOINT.
     - e. SEE WALL PANEL SHEETS FOR ADDITIONAL REQUIREMENTS.

8. **ELECTRICAL SYSTEM**
   - A. INSTALLATION AND WIRING:
     - a. SHALL CONFORM TO THE REQUIREMENTS OF THE NATIONAL ELECTRICAL CODE PER DRAWING G0.1.
     - b. SYSTEMS SHALL BE INSTALLED PER MANUFACTURER'S WRITTEN INSTRUCTIONS.

9. **MECHANICAL SYSTEM(S)**
   - A. INSTALLATION AND WIRING:
     - a. WHEN SUPPLIED, SHALL CONFORM TO THE REQUIREMENTS OF THE APPLICABLE CODE(S) PER DRAWING G0.1.
     - b. SYSTEMS SHALL BE INSTALLED PER MANUFACTURER'S WRITTEN INSTRUCTIONS.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(100 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LONNIE A MARVEL LICENSE No. 83750 STATE OF FLORIDA PROFESSIONAL ENGINEER

2024.05.29 15:26:32-04'00'

No	Revision Description	Date	By

MODULAR CONNECTIONS

1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565
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12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site

IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name

GENERAL SPECIFICATIONS

Project Number:

Sheet Number

G0.3

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one half inch = one foot (0, 4)

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GENERAL SYMBOL LEGEND

ABBREVIATIONS

&	AND
@	AT
∠	ANGLE
℄	CENTER LINE
°	DEGREE
Ø	DIAMETER
=	EQUAL TO
'	FEET
"	INCHES
¼	FRACTION ONE QUARTER
>	GREATER THAN
≥	GREATER-THAN OR EQUAL TO
<	LESS-THAN
≤	LESS-THAN OF EQUAL TO
-	MINUS
#	NUMBER OR POUNDS
%	PERCENT
±	PLUS OR MINUS
+	PLUS
BOT.	BOTTOM
CONC.	CONCRETE
CONT.	CONTINUOUS
CLR.	CLEAR
DTL.	DETAIL
DWG.	DRAWING
EQ.	EQUAL
EW	ENDWALL
GALV.	GALVINIZED
HDG	HOT DIPPED GALVANIZED
I.H.	INTERIOR HEIGHT
MAX.	MAXIMUM
MIN.	MINIMUM
MISC.	MISCELLANEOUS
MPH	MILES PER HOUR
MIR.	MIRROR
O.C.	ON CENTER
PNL	PANEL
PSF	POUNDS PER SQUARE FEET
PSI	POUNDS PER SQUARE INCH
REINF.	REINFORCING
SCH.	SCHEDULE
SIM.	SIMILAR
STL.	STEEL
SW	SIDEWALL
T&B	TOP AND BOTTOM
TYP.	TYPICAL

SYMBOLS

SYMBOL	DESCRIPTION
B / A — PLAN NORTH — C / D; KEYPLAN	WALL PANEL DESIGNATION; PLAN NORTH DESIGNATION
	BREAK LINE
	ELEVATION TAG
1 View Name 1/8" = 1'-0"	PLAN TITLE
	SPAN DIRECTION
1t	PANEL TAG
123 OR 1	ITEM TAG
0	GRID BUBBLE
Room name 101	ROOM TAG
SECTION REFERENCE, DIRECTION INDICATOR, 1, SIM, A101, SHEET REFERENCE	SECTION MARKER
DETAIL OR SECTION REFERENCE, 1, SIM, A101, SHEET REFERENCE; ENCLOSURE AROUND ITEM TO BE DETAILED	DETAIL CALLOUT BUBBLE
VIEW DESIGNATION; Ref View Name; A101; SHEET REFERENCE; DIRECTION INDICATOR; Ref View Name	ELEVATION MARKER

NOTE: SOME SYMBOLS AND/OR ABBREVIATIONS SHOWN ON THIS LEGEND MAY NOT PERTAIN TO THIS PROJECT

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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
ABBREVIATIONS AND SYMBOLS

Project Number: Sheet Number G0.4

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
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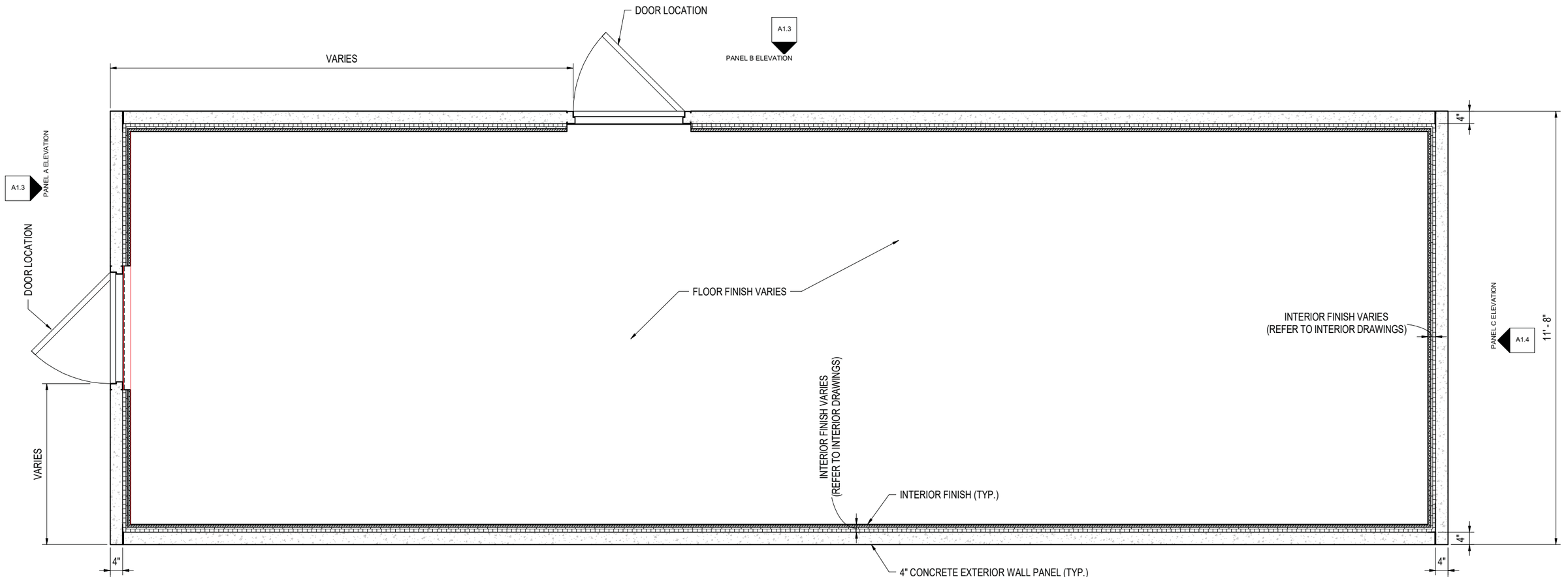
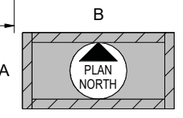


1 FLOOR PLAN
3/4" = 1'-0"

NOTES:
1. COORDINATE WITH TRADE DRAWINGS FOR EXACT PENETRATION LOCATIONS.
2. REFER TO ALL TRADES FOR ADDITIONAL INFORMATION AND COORDINATION.



KEYPLAN

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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
FLOOR PLAN

Project Number: Sheet Number
A1.1

one quarter inch = one foot (0, 4, 8)
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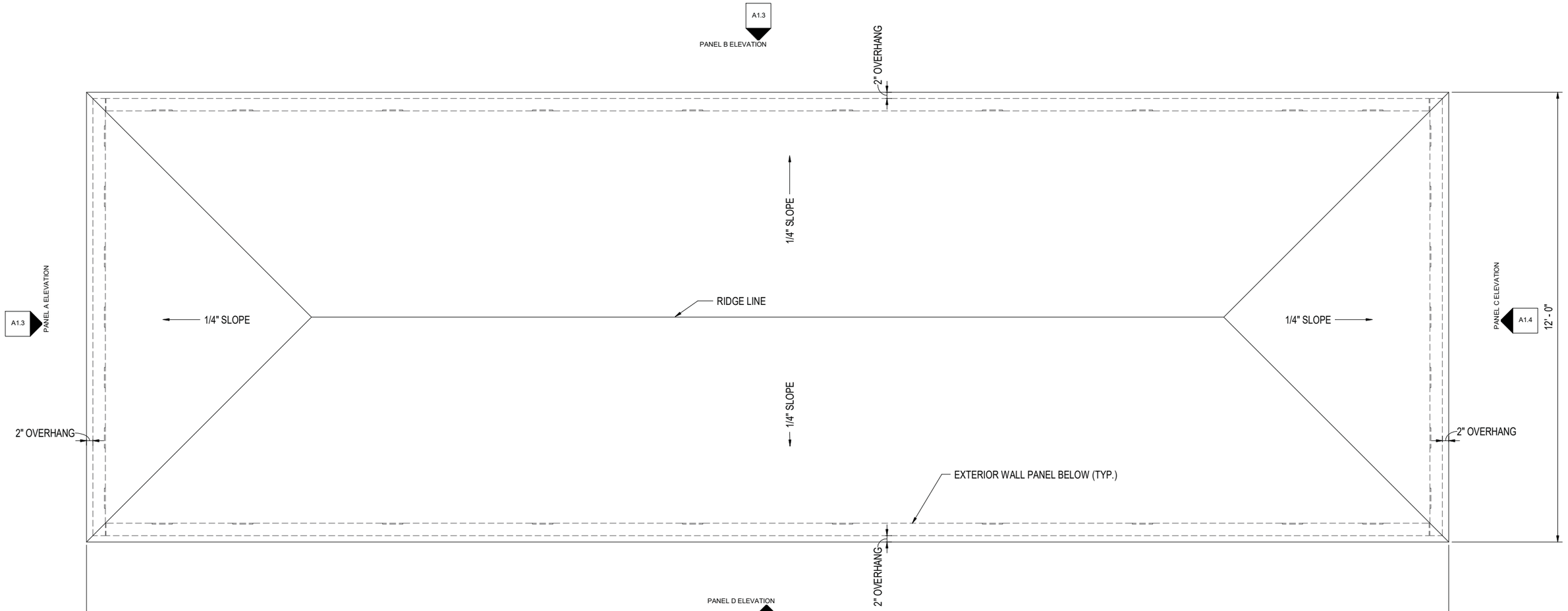

1 ROOF PLAN
3/4" = 1'-0"

NOTES:
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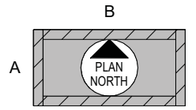
2. REFER TO ALL TRADES FOR ADDITIONAL INFORMATION AND COORDINATION.



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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
ROOF PLAN

Project Number: Sheet Number
A1.2

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
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one and one half inch = one foot (0, 1)

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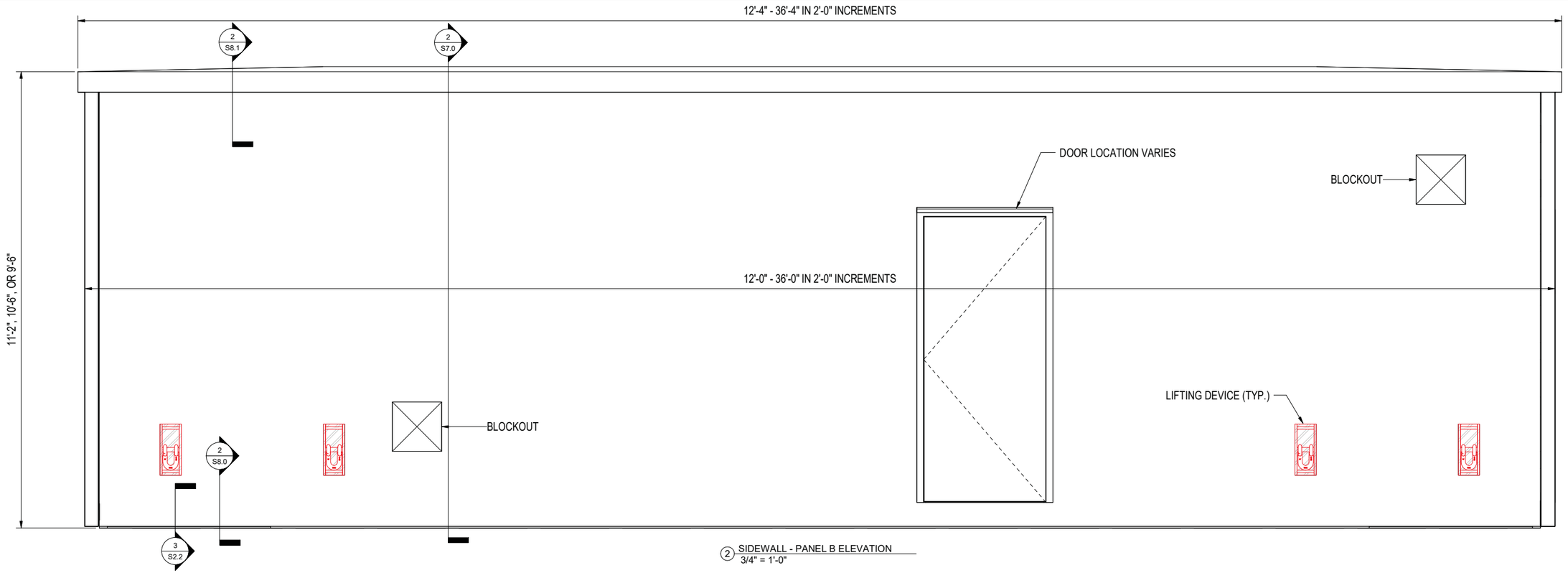
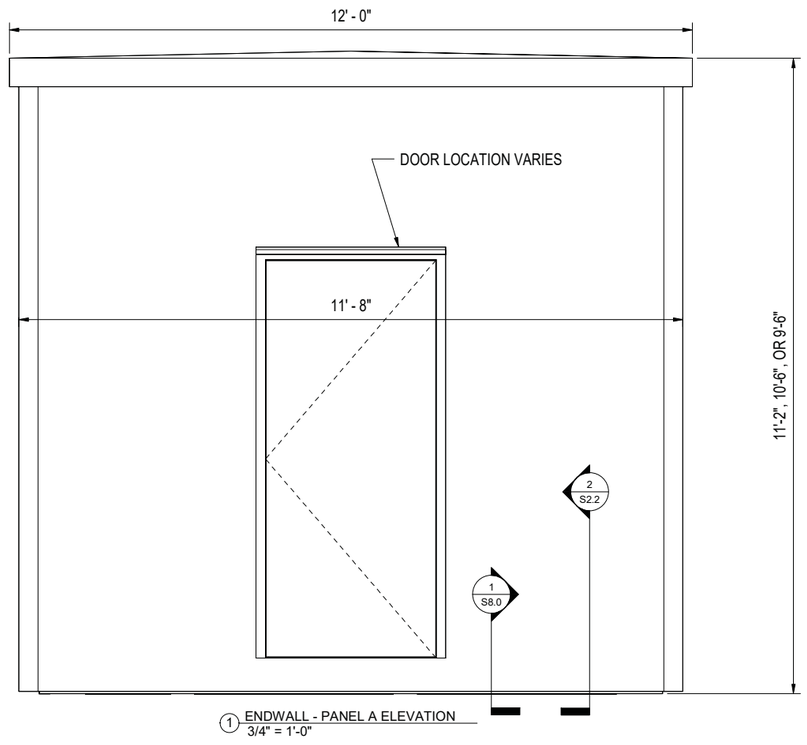


2 SIDEWALL - PANEL B ELEVATION
3/4" = 1'-0"



1 ENDWALL - PANEL A ELEVATION
3/4" = 1'-0"

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Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

NOTES:
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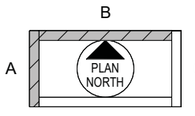

KEYPLAN

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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
EXTERIOR ELEVATIONS - PANEL A & B

Project Number:

Sheet Number
A1.3

one quarter inch = one foot (0, 4, 8)
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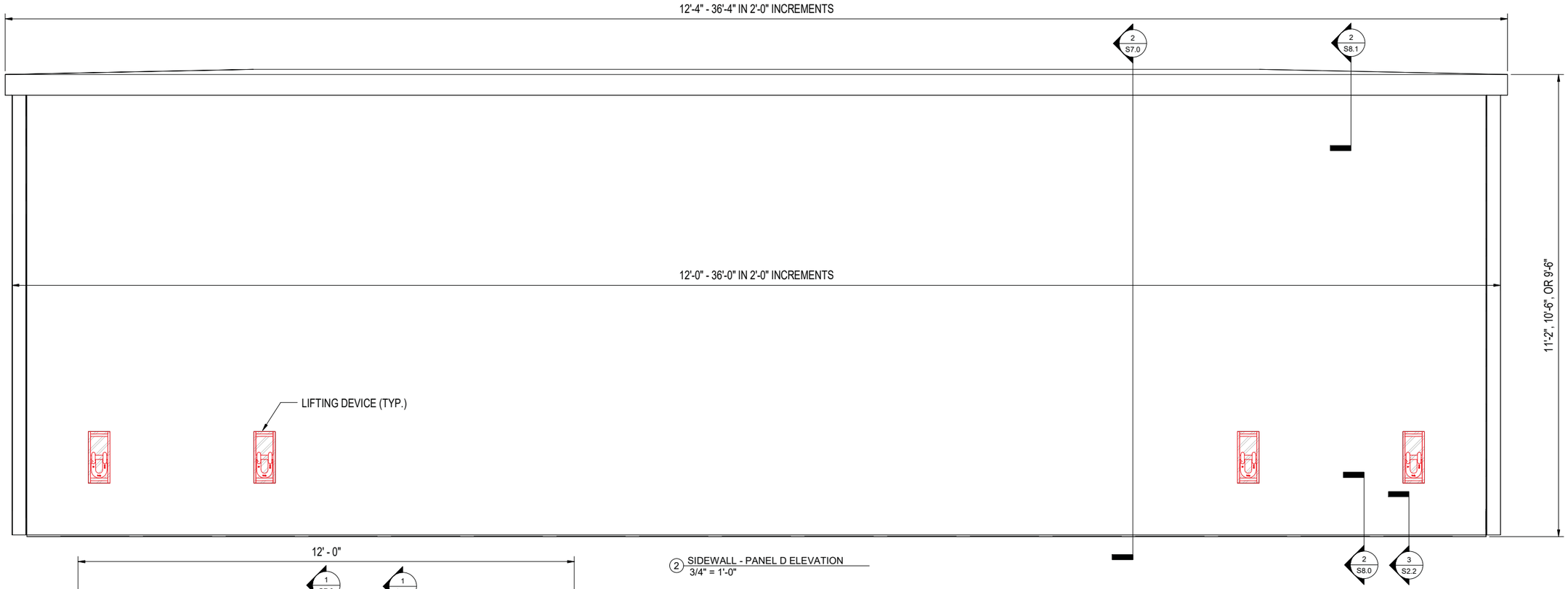

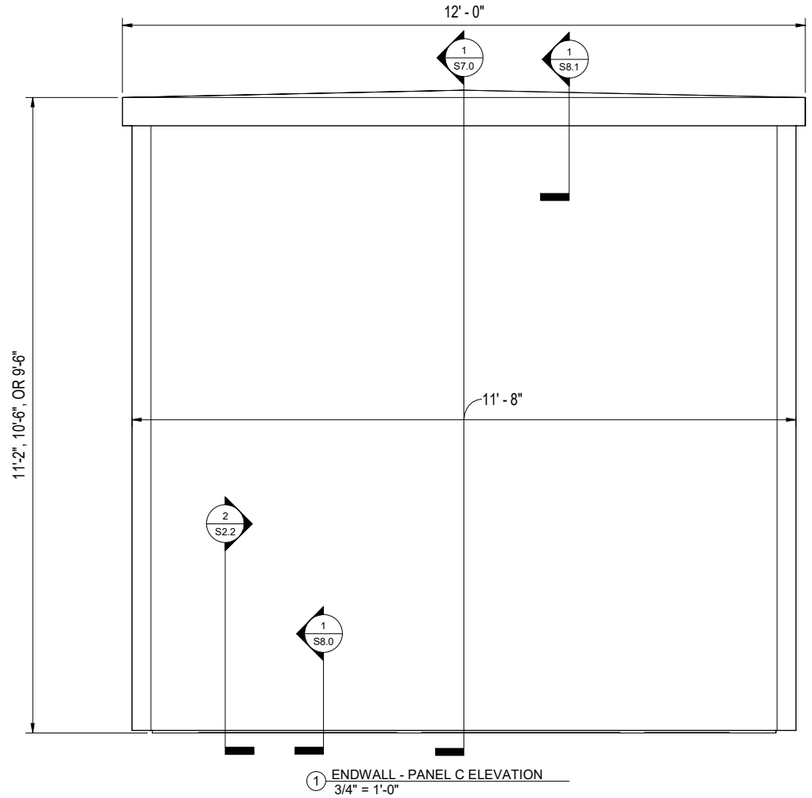

1 ENDWALL - PANEL C ELEVATION
3/4" = 1'-0"

2 SIDEWALL - PANEL D ELEVATION
3/4" = 1'-0"

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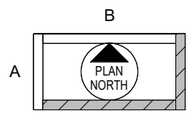

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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
EXTERIOR ELEVATIONS - PANEL C & D

Project Number: Sheet Number
A1.4

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three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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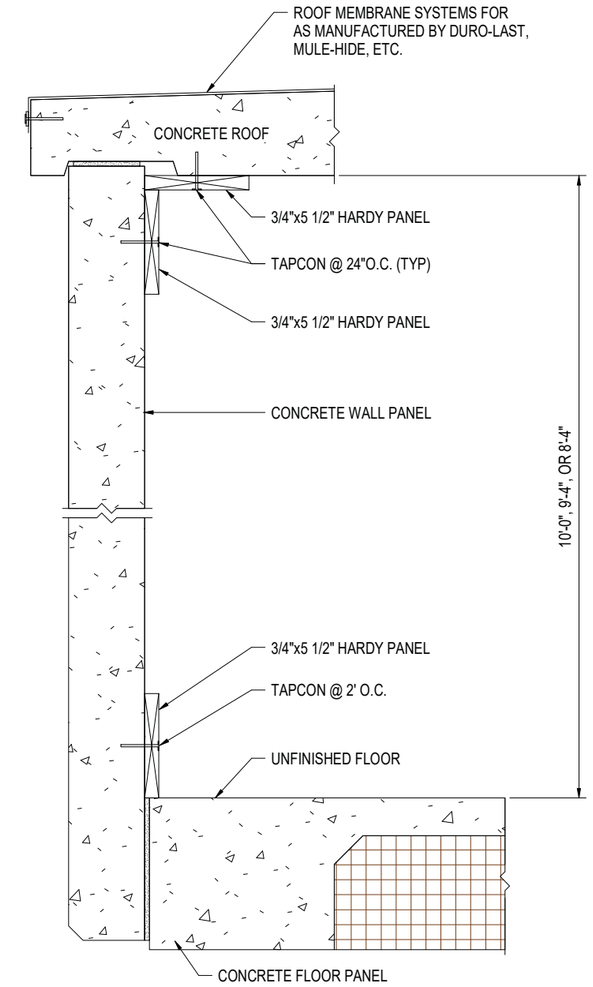
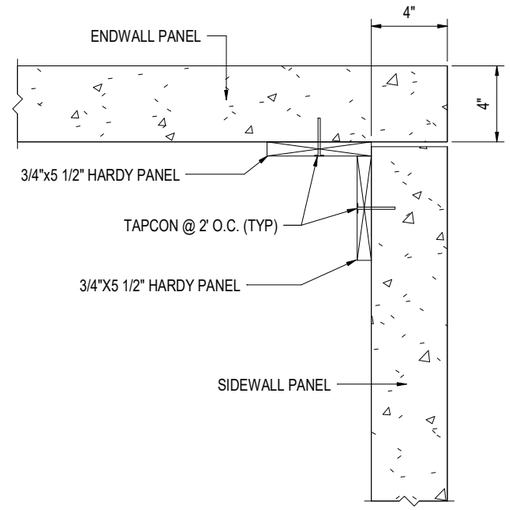


1 WALL SECTION - INTERIOR FINISH
3" = 1'-0"



2 CORNER DETAIL - INTERIOR FINISH
3" = 1'-0"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

NOTES:

1. THE INSULATION CAST IN THE FLOOR IS 6" THICK EXPANDED POLYSTYRENE BOARD, 1 LB. DENSITY (R-3.85 PER 1"), WHICH WILL GIVE THE FOLLOWING R-VALUES FOR EACH WIDTH BUILDING:
   A. 12' TO 28' LENGTH BUILDINGS USE R-14.80 (U = 0.0676)
   B. 30' TO 36' LENGTH BUILDINGS USE R-15.85 (U = 0.0631)
2. ANY WOOD IN CONTACT WITH THE CONCRETE SHALL BE PRESSURE TREATED UTILITY GRADE OR BETTER. WALL FURRING STRIPS SHALL BE UTILITY GRADE SPF OR BETTER.
3. WALL R-VALUE = R2.73.
4. ROOF R-VALUES = OPTION #1 R3.63, OPTION #2 R3.318, OPTION #3 R2.73.
5. UNCONDITIONED SPACE SHALL HAVE HARDY PANEL INSTALLED AT ALL PANEL JOINTS AND PAINTED.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
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No.	Revision Description	Date	By

MODULAR CONNECTIONS
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565
Email: info@modularconnections.com

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Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
INTERIOR FINISH & MISC. DETAILS - OPTION A

Project Number:

Sheet Number
A2.0A

one quarter inch = one foot
0 4 8

three eighths inch = one foot
0 4

one half inch = one foot
0 4

three quarters inch = one foot
0 2

one and one half inch = one foot
0 1

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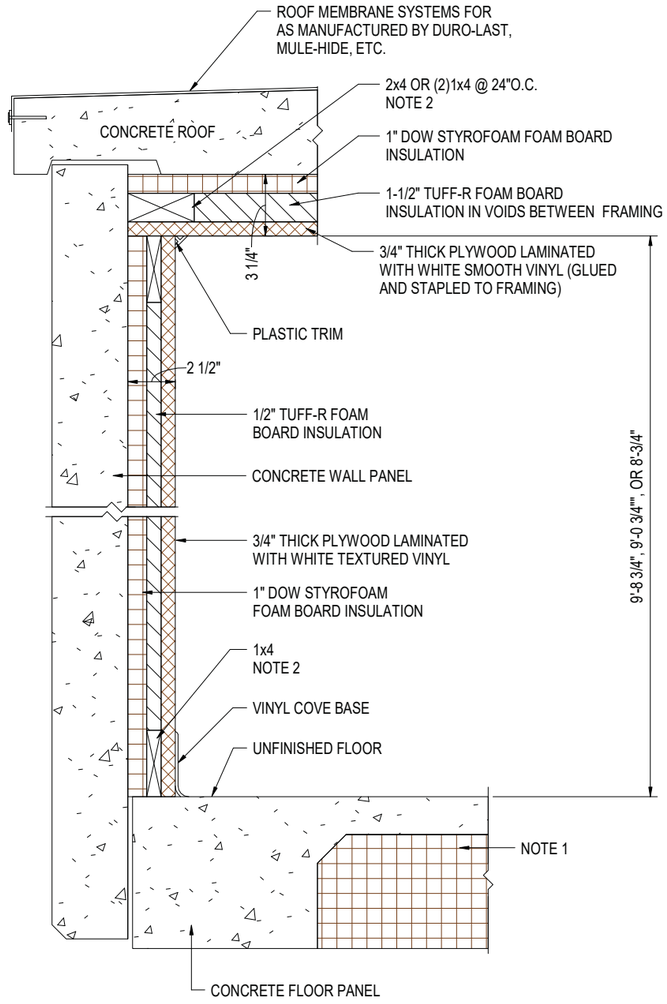

1 WALL SECTION - INTERIOR FINISH
3" = 1'-0"

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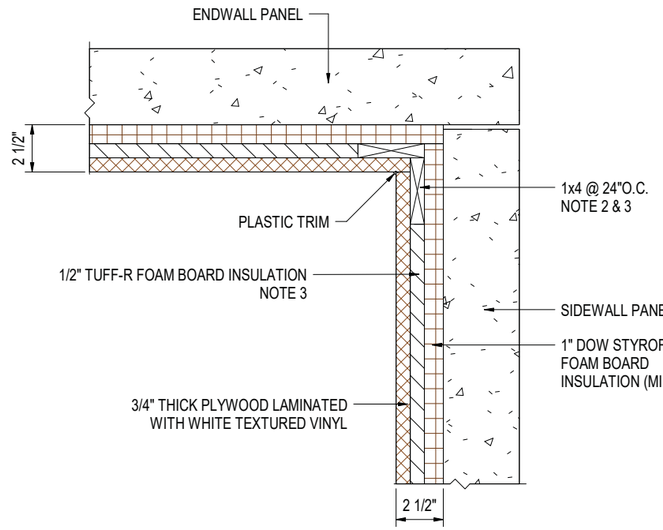
APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S3
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

INTERIOR FINISH FASTENER SCHEDULE

LOCATION	MEMBER	FASTENING
CEILING	2x4	16d x 2 1/4" GALV. NAILS @ 12"O.C.
WALL	1x4	(1) 16d x 2 1/4" NAILS @ 12"O.C.
	PLYWOOD	CONSTRUCTION ADHESIVE TO MEET APA AFG-01 AND USE 16 GA. x 2" STAPLES @ 10"O.C. @ PERIMETER
FLOOR	BROOM, PAINTED, OR VINYL FINISH	PER MANUFACTURER

NOTE:
ANY WALL MOUNTED DEVICE WEIGHING OVER 60 LBS. WILL BE ATTACHED TO THE CONCRETE WALL W/ A MINIMUM OF (4) 1/4" TAPCONS W/ A MINIMUM CONCRETE PENETRATION OF 1 3/4" IN DEPTH.



2 CORNER DETAIL - INTERIOR FINISH
3" = 1'-0"

NOTES:

1. THE INSULATION CAST IN THE FLOOR IS 6" THICK EXPANDED POLYSTYRENE BOARD, 1 LB. DENSITY (R-3.85 PER 1"), WHICH WILL GIVE THE FOLLOWING R-VALUES FOR EACH WIDTH BUILDING:
   A. 12' TO 28' LENGTH BUILDINGS USE R-15.74 (U = 0.0635)
   B. 30' TO 36' LENGTH BUILDINGS USE R-16.55 (U = 0.0604)
2. ANY WOOD IN CONTACT WITH THE CONCRETE SHALL BE PRESSURE TREATED UTILITY GRADE OR BETTER. WALL FURRING STRIPS SHALL BE UTILITY GRADE SPF OR BETTER.
3. WALL R-VALUE: R13.3 (U = 0.075).
4. ROOF R-VALUES: OPTION #1 = R22.12, OPTION #2 = R21.67, OPTION #3 = R21.22.

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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
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Drawing Number:

Sheet Name
INTERIOR FINISH & MISC. DETAILS - OPTION B

Project Number:

Sheet Number
A2.0B

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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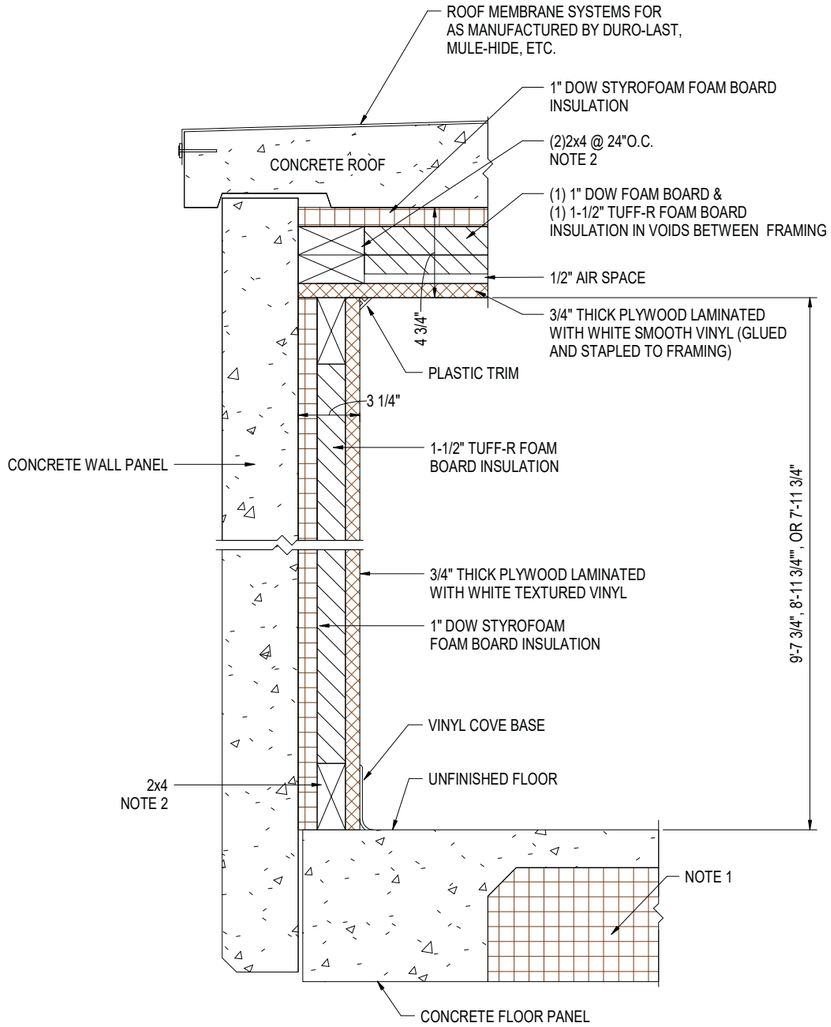


1 WALL SECTION - INTERIOR FINISH
3" = 1'-0"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

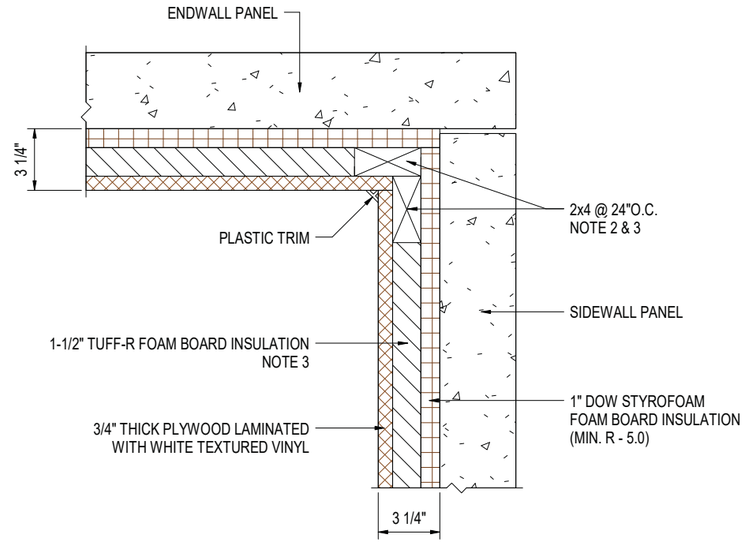
APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S3
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

INTERIOR FINISH FASTENER SCHEDULE

LOCATION	MEMBER	FASTENING
CEILING	2x4	16d x 2 1/4" GALV. NAILS @ 12"O.C.
WALL	2x4	(1) 16d x 3 1/4" NAILS @ 12"O.C.
	PLYWOOD	CONSTRUCTION ADHESIVE TO MEET APA AFG-01 AND USE 16 GA. x 2" STAPLES @ 10"O.C. @ PERIMETER
FLOOR	BROOM, PAINTED, OR VINYL FINISH	PER MANUFACTURER

NOTE:
ANY WALL MOUNTED DEVICE WEIGHING OVER 60 LBS. WILL BE ATTACHED TO THE CONCRETE WALL W/ A MINIMUM OF (4) 1/4" TAPCONS W/ A MINIMUM CONCRETE PENETRATION OF 1 3/4" IN DEPTH.



2 CORNER DETAIL - INTERIOR FINISH
3" = 1'-0"

NOTES:

1. THE INSULATION CAST IN THE FLOOR IS 6" THICK EXPANDED POLYSTYRENE BOARD, 1 LB. DENSITY (R-3.85 PER 1"), WHICH WILL GIVE THE FOLLOWING R-VALUES FOR EACH WIDTH BUILDING:
   A. 12' TO 28' LENGTH BUILDINGS USE R-16.08 (U = 0.0622)
   B. 30' TO 36' LENGTH BUILDINGS USE R-16.80 (U = 0.0595)
2. ANY WOOD IN CONTACT WITH THE CONCRETE SHALL BE PRESSURE TREATED UTILITY GRADE OR BETTER. WALL FURRING STRIPS SHALL BE UTILITY GRADE SPF OR BETTER.
3. WALL R-VALUE: R19.9 (U = 0.05).
4. ROOF R-VALUES: OPTION #1 = R27.12, OPTION #2 = R26.67, OPTION #3 = R26.22.

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Site
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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
INTERIOR FINISH & MISC. DETAILS - OPTION C

Project Number:

Sheet Number
A2.0C

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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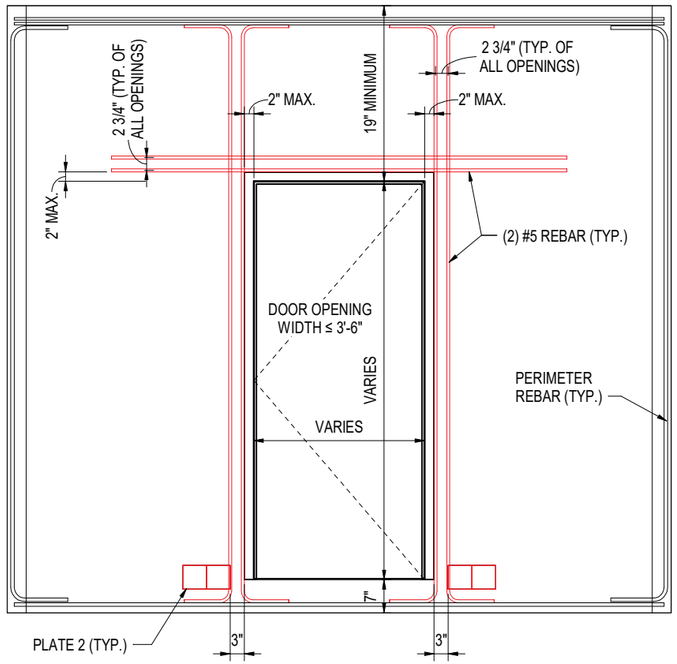
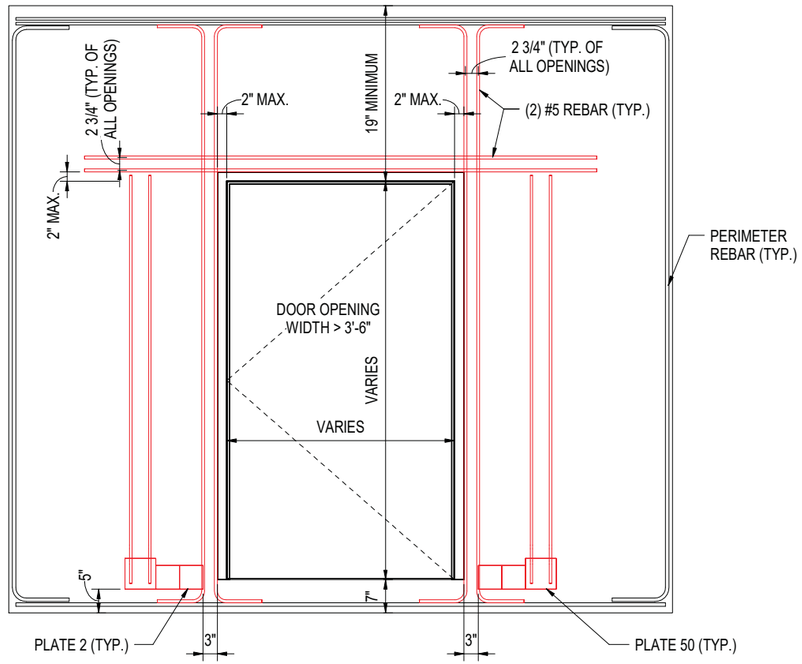
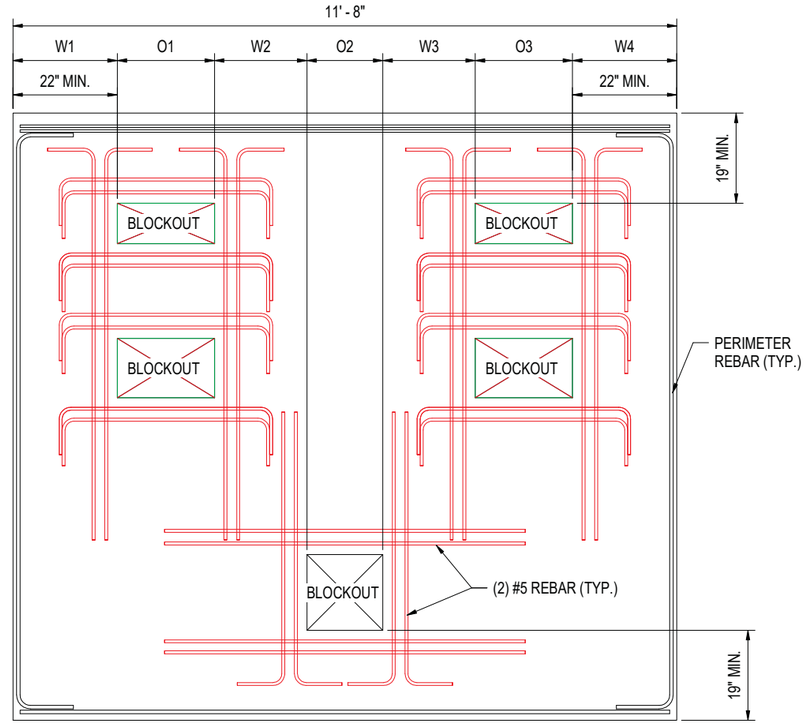


NOTE:
DOOR OPENINGS 3'-6" AND LESS SHALL HAVE (2)PLATE 2'S ON EACH SIDE OF OPENING AS INDICATED.



NOTE:
DOOR OPENINGS GREATER THAN 3'-6" SHALL HAVE (1)PLATE 50 & (2)PLATE 2'S ON EACH SIDE OF OPENING AS INDICATED.



NOTE:
THE TOTAL WIDTH OF OPENINGS ALONG ANY HORIZONTAL PLANE SHALL NOT EXCEED THE TOTAL NET WIDTH OF CONCRETE.

(W1+W2+W3+W4 ≥ O1+O2+O3)

③ OPENING REINFORCEMENT AND NET CONCRETE REQUIREMENTS
3/4" = 1'-0"

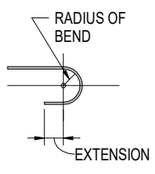
① DOOR OPENINGS 3'-6" OR LESS
3/4" = 1'-0"

② DOOR OPENINGS GREATER THAN 3'-6"
3/4" = 1'-0"

STEEL REINFORCEMENT

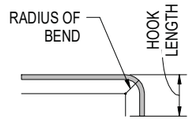
180 ° HOOKED BARS

BAR	RADIUS OF BEND	EXTENSION
3	1 1/2"	2 1/2"
4	2"	2 1/2"
5	2 1/2"	2 1/2"
6	3"	3"
7	3 1/2"	3 1/2"
8	4"	4"



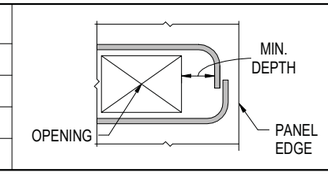
STANDARD 90° HOOK

BAR	HOOK	RADIUS OF BEND
#3	6"	1 1/2"
#4	8"	2"
#5	10"	2 1/2"
#6	12"	3"
#7	14"	3 1/2"
#8	16"	4"

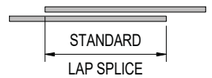


MINIMUM EMBEDMENT FOR STANDARD 90° HOOK

BAR	DEPTH
#4	9"
#5	11"
#6	13"
#7	15"



STANDARD BAR LAP SPLICE

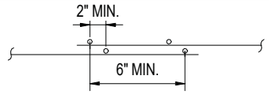


CLR - INDICATES AMOUNT OF CLEAR COVER

db - INDICATES BAR DIA.

	BAR	#3	#4	#5	#6	#7	#8
LENGTH	CLR ≤ db	18"	26"	40"	57"	77"	101"
	db < CLR < 2db	18"	24"	30"	40"	54"	71"
	2db ≤ CLR	18"	24"	30"	36"	42"	51"

STANDARD WIRE LAP SPLICE



MINIMUM CONCRETE COVER

CONCRETE EXPOSED TO EARTH OR WEATHER:	
WALL PANELS	3/4"
OTHERS	1 1/2"
CONCRETE NOT EXPOSED TO EARTH OR WEATHER:	
SLABS, WALLS & JOISTS	3/4"
BEAMS	1"
STIRRUPS	1/2"

NOTES:
1. MAXIMUM DOOR OPENING/BLCOKOUT WIDTH IS 6'-0".
2. REFER TO S4 DRAWINGS FOR ADDITIONAL CLEARANCE INFORMATION.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
OPENING REINFORCEMENT DETAILS AND LEGEND

Project Number:

Sheet Number
S0.1

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three eighths inch = one foot
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one and one half inch = one foot
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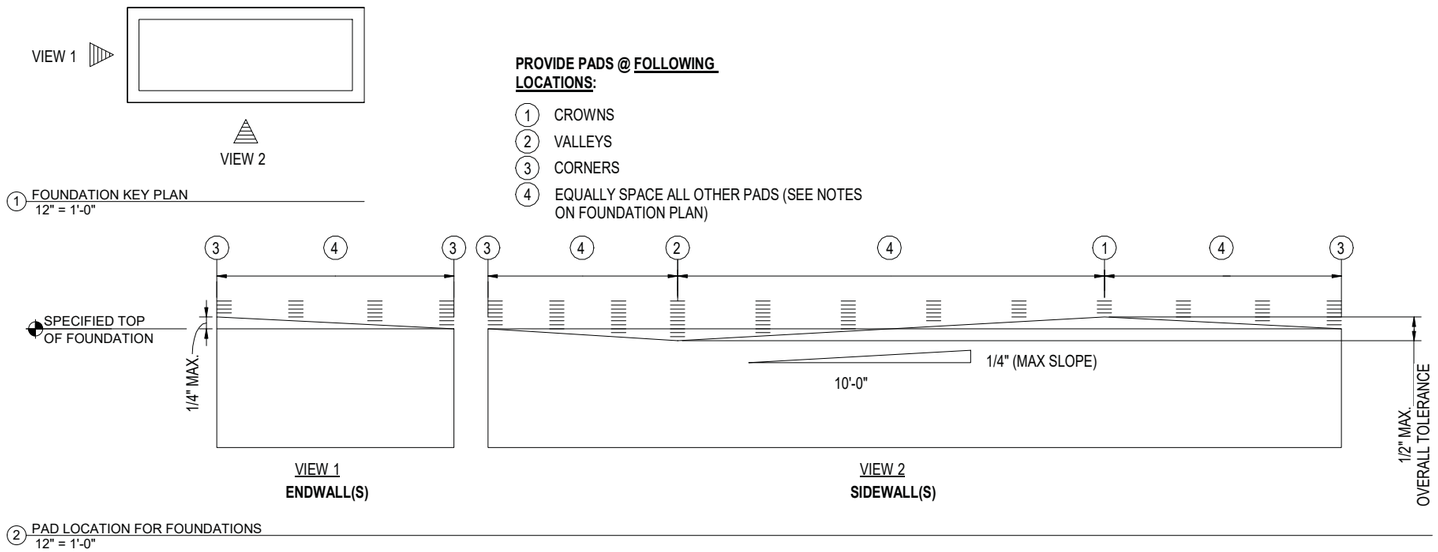

1 FOUNDATION KEY PLAN
12" = 1'-0"

2 PAD LOCATION FOR FOUNDATIONS
12" = 1'-0"

FOUNDATION GENERAL NOTES:

A. GEOTECHNICAL INFORMATION HAS NOT BEEN PROVIDED. THE FOUNDATION DESIGN ASSUMES MINIMUM ALLOWABLE SOIL BEARING PRESSURE OF **2500 PSF.** SITE CLASS **D**.
   a. FIELD VERIFICATION SHALL BE PERFORMED PRIOR TO COMMENCEMENT OF FOUNDATION CONSTRUCTION. IF ALLOWABLE SOIL BEARING PRESSURE IS FOUND TO BE LESS THAN STATED ABOVE, NOTIFY THE ENGINEER OF RECORD IMMEDIATELY.
   b. IF ANY GEOTECHNICAL INFORMATION IS AVAILABLE, FORWARD TO THE ENGINEER OF RECORD IMMEDIATELY.
B. WORK SHALL BE IN ACCORDANCE WITH LOCAL CODES, SAFETY REGULATIONS AND UNLESS OTHERWISE NOTED, THE LATEST REVISION OF ACI 318, "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE". PROCEDURES FOR THE PROTECTION OF EXCAVATIONS, EXISTING CONSTRUCTION AND UTILITIES SHALL BE ESTABLISHED PRIOR TO FOUNDATION INSTALLATION.
C. CONCRETE MATERIALS SHALL CONFORM TO THE APPROPRIATE STATE REQUIREMENTS FOR EXPOSED STRUCTURAL CONCRETE.
D. PROPORTIONS OF CONCRETE MATERIALS SHALL BE SUITABLE FOR THE INSTALLATION METHOD UTILIZED AND SHALL RESULT IN DURABLE CONCRETE FOR RESISTANCE TO LOCAL ANTICIPATED AGGRESSIVE ACTIONS. THE DURABILITY REQUIREMENTS OF ACI 318 CHAPTER 4 SHALL BE SATISFIED BASED ON THE CONDITIONS EXPECTED AT THE SITE. AS A MINIMUM, CONCRETE SHALL DEVELOP A MINIMUM COMPRESSIVE STRENGTH OF 4000 PSI IN 28 DAYS.
E. MAXIMUM SIZE OF CONCRETE AGGREGATE SHALL NOT EXCEED 1 INCH; SIZE SUITABLE FOR INSTALLATION METHOD UTILIZED; OR ONE-THIRD CLEAR DISTANCE BEHIND OR BETWEEN REINFORCING.
F. REINFORCEMENT SHALL BE DEFORMED AND CONFORM TO THE REQUIREMENTS OF ASTM A615 GRADE 60 UNLESS OTHERWISE NOTED.
G. WELDING IS PROHIBITED ON REINFORCING STEEL AND EMBEDMENTS.
H. MINIMUM CONCRETE COVER FOR REINFORCEMENT SHALL BE 3 INCHES UNLESS OTHERWISE NOTED.
I. CONCRETE COVER FROM TOP OF FOUNDATION TO ENDS OF VERTICAL REINFORCEMENT SHALL NOT EXCEED 3 INCHES NOR BE LESS THAN 2 INCHES.
J. ALL HORIZONTAL BARS IN WALLS & BEAM EDGES SHALL BE BENT AT CORNERS IN SUCH A WAY THAT CONTINUITY IS PROVIDED THROUGH THE JOINT. SEPARATE CORNER BARS OF THE SAME SIZE AND SPACING AS THE HORIZONTAL REINFORCING MAY BE SUBSTITUTED FOR THE BENT PORTION OF THE CONTINUOUS BARS.
K. FOUNDATION DESIGN ASSUMES STRUCTURAL BACKFILL TO BE COMPACTED IN 8 INCH MAXIMUM LAYERS TO 95% OF MAXIMUM DRY DENSITY AT OPTIMUM MOISTURE CONTENT IN ACCORDANCE WITH ASTM D698. ADDITIONALLY, STRUCTURAL BACKFILL MUST HAVE A MINIMUM COMPACTED UNIT WEIGHT OF 100 POUNDS PER CUBIC FOOT.
L. FOUNDATION INSTALLATIONS SHALL BE SUPERVISED BY PERSONNEL KNOWLEDGEABLE AND EXPERIENCED WITH THE PROPOSED FOUNDATION TYPE. CONSTRUCTION SHALL BE IN ACCORDANCE WITH GENERALLY ACCEPTED INSTALLATION PRACTICES.
M. FOUNDATION DESIGN ASSUMES FIELD INSPECTIONS WILL BE PERFORMED TO VERIFY THAT CONSTRUCTION MATERIALS, INSTALLATION METHODS AND ASSUMED DESIGN PARAMETERS ARE ACCEPTABLE BASED ON CONDITIONS EXISTING AT THE SITE.
N. LOOSE MATERIAL SHALL BE REMOVED FROM BOTTOM OF EXCAVATION PRIOR TO CONCRETE PLACEMENT.
O. CONCRETE SHALL BE PLACED IN A MANNER THAT WILL PREVENT SEGREGATION OF CONCRETE MATERIALS, INFILTRATION OF WATER OR SOIL AND OTHER OCCURRENCES WHICH MAY DECREASE THE STRENGTH OR DURABILITY OF FOUNDATION.
P. CONCRETE SHALL BE PLACED AGAINST UNDISTURBED SOIL. WHEN FORMS ARE NECESSARY, THEY SHALL BE REMOVED PRIOR TO PLACING STRUCTURAL BACKFILL.
Q. FOUNDATION DESIGN ASSUMES CONTINUOUS CONCRETE PLACEMENT WITHOUT CONSTRUCTION JOINTS.
R. FOOTING ELEVATIONS INDICATED ON THE DOCUMENTS ARE DESIGNED TO MEET ANTICIPATED DEPTHS FOR ALLOWABLE BEARING CAPACITY AND/OR DEPTHS REQUIRED FOR FROST PROTECTION. ACTUAL FIELD CONDITIONS ENCOUNTERING UNSUITABLE SOILS MAY REQUIRE ADDITIONAL EXCAVATION TO LOWER FOOTING ELEVATIONS OR REPLACE WITH STRUCTURAL FILL. CONTRACTOR SHALL PROMPTLY REPORT THESE CONDITIONS BEFORE PROCEEDING WITH ADDITIONAL WORK.
S. VAPOR BARRIER SHALL BE PROVIDED BETWEEN THE SUBGRADE AND BOTTOM OF FLOOR IN A LOCATION SUITABLE FOR SITE CONDITIONS.

Per Local Approval and Inspection

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER
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No.	Revision Description	Date	By

MODULAR CONNECTIONS
1090 Industrial Blvd.
Bessemer, AL 35022
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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
FOUNDATION NOTES & INFORMATION

Project Number:

Sheet Number
S1.0

one quarter inch = one foot
0 4 8

three eighths inch = one foot
0 4

one half inch = one foot
0 4

three quarters inch = one foot
0 2

one and one half inch = one foot
0 1

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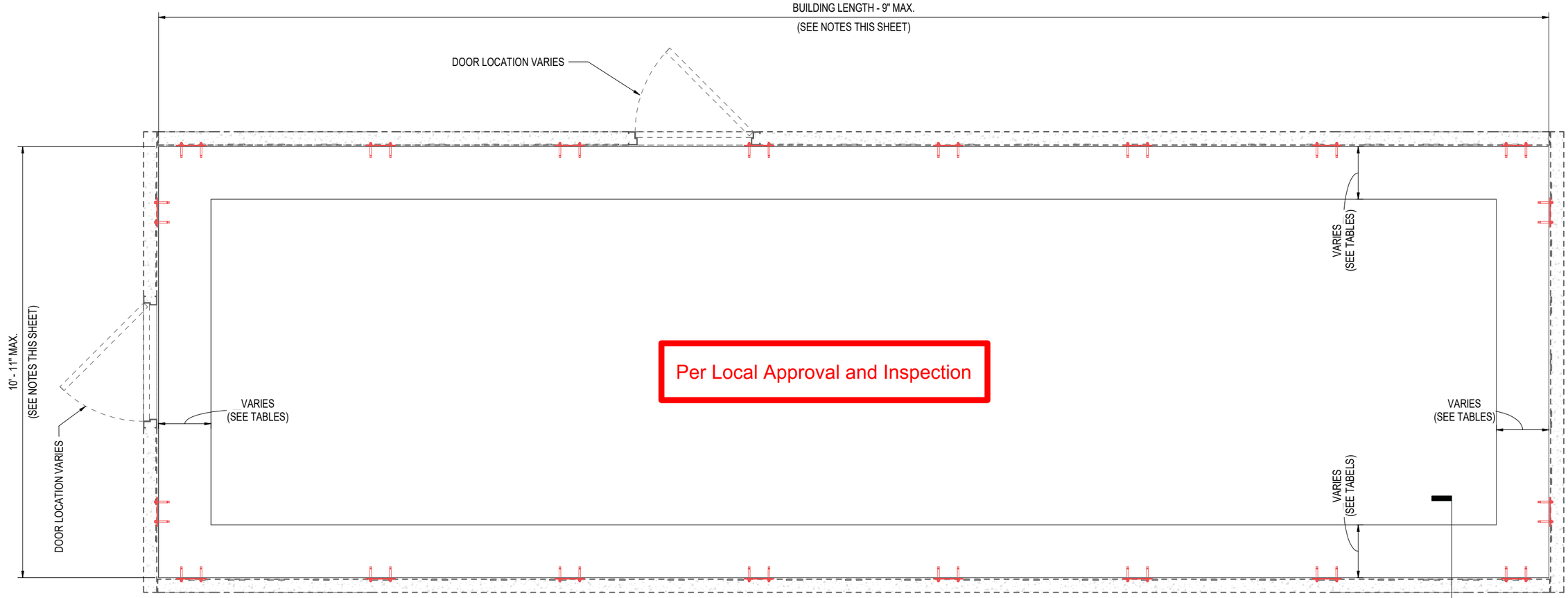


1 FOUNDATION PLAN (PERIMETER BEAM)
3/4" = 1'-0"

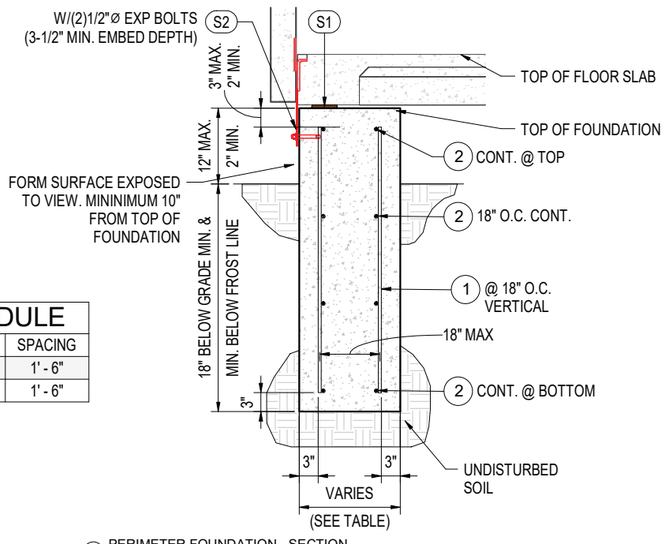
NOTES:

1. EITHER FOUNDATION TYPE, PERIMETER BEAM OR SLAB ON GRADE, MAY BE UTILIZED, CONTRACTORS PREFERENCE. CONTRACTOR SHALL COORDINATE WITH OWNER.
2. REEFER TO BEARING PAD & LATERAL SLIDE PLATE PLAN FOR ADDITIONAL INFORMATION.
3. UNLESS NOTED OTHERWISE, FOUNDATION SHALL REST ON UNDISTURBED SOIL OR COMPACTED STRUCTURAL FILL.
4. FIELD VERIFY ALLOWABLE BEARING CAPACITY. CONTRACTOR SHALL PROMPTLY REPORT BEARING CAPACITIES LESS THAN STATED IN GENERAL FOUNDATION NOTES BEFORE PROCEEDING WITH ADDITIONAL WORK.
5. PERIMETER BEAM DEPTH - SEE DETAIL THIS SHEET.
6. TOP OF FOUNDATION ELEVATION TOLERANCE 1/4" IN 10'-0" & 1/2" MAX. OVERALL.
   a. SHIM W/ 4x4 NEOPRENE BEARING PADS TO ATTAIN SAME ELEVATION WITHIN 1/16"(±). SEE DRAWING S1.6 FOR ADDITIONAL INFORMATION.
7. FOUNDATION WIDTH AND LENGTH SHALL NOT EXCEED DIMENSIONS SHOWN.
   a. FOUNDATION WIDTH AND LENGTH SHALL NOT BE LESS THAN 1/4" LESS THAN DIMENSIONS SHOWN.
8. PROVIDE 12 MIL VAPOR BARRIER WITH 6" LAPPED & TAPED JOINTS BETWEEN SUBGRADE & BUILDING FLOOR.

PERIMETER BEAM			
ROOF LIVE LOAD (PSF)	FLOOR LIVE LOAD (PSF)		
	150	200	250
35	12"	14"	14"
80	12"	14"	14"
100	14"	14"	15"
NOTE: WIDTHS ARE IN INCHES			

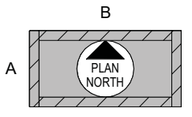
3 PERIMETER BEAM FOUNDATION WIDTH SCHEDULE
12" = 1'-0"

REBAR SCHEDULE			
ITEM #	TYPE	SHAPE	SPACING
1	#4	00	1' - 6"
2	#5	00	1' - 6"



2 PERIMETER FOUNDATION - SECTION
1" = 1'-0"

PARTS SCHEDULE - FOUNDATION	
ITEM #	DESCRIPTION
	ANCHOR, 1/2"-13 X 4-1/2" WEDGE ANCHOR STUD HDG HILTI
S1	BEARING PAD, 1/4" X 4" X 4" NEOPRENE
S2	LATERAL SLIDE PLATE, HDG



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LICENSE
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PROFESSIONAL ENGINEER
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No	Revision Description	Date	By

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IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
PERIMETER BEAM FOUNDATION PLAN & DETAILS

Project Number:

Sheet Number
S1.1A

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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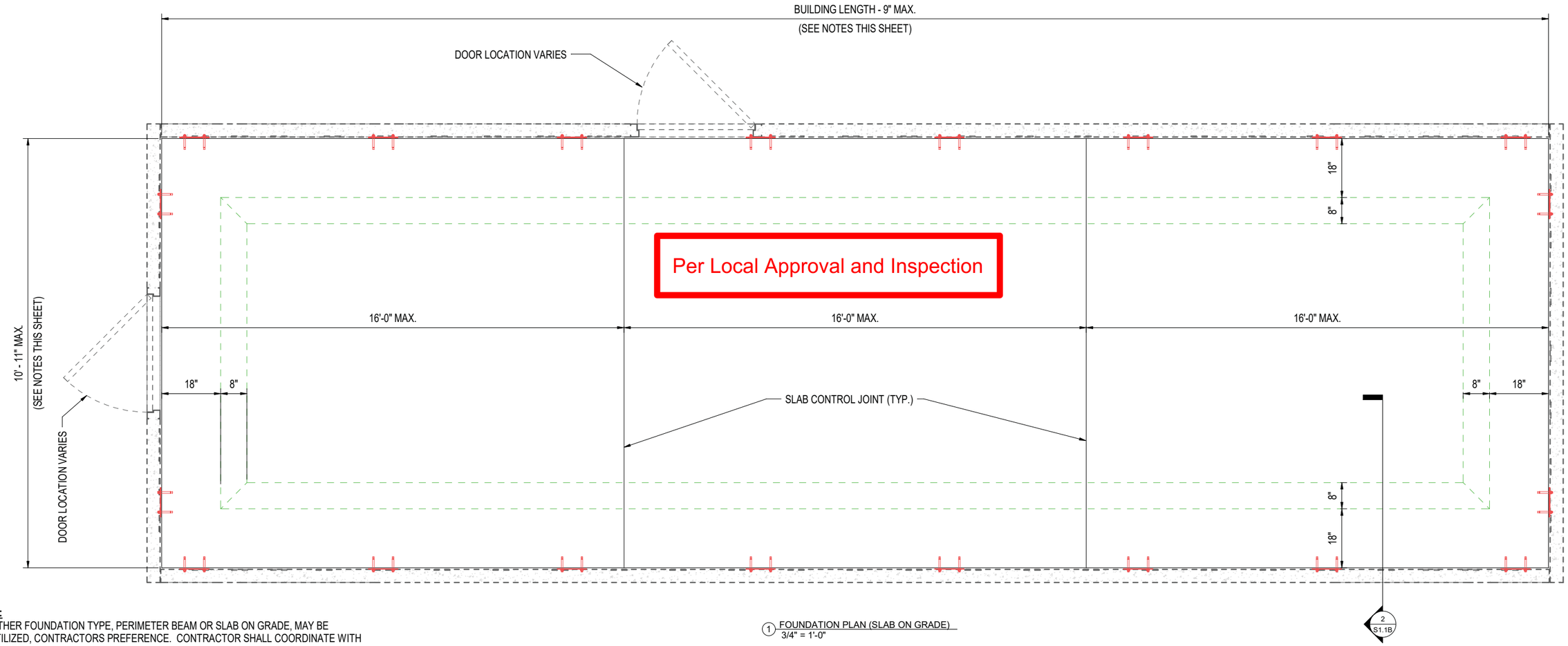
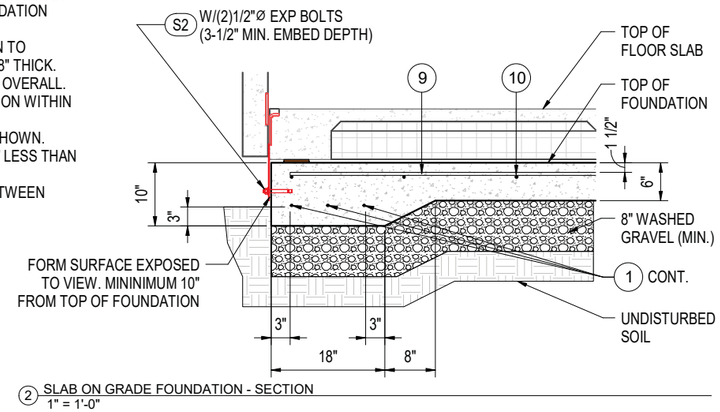


NOTES:

1. EITHER FOUNDATION TYPE, PERIMETER BEAM OR SLAB ON GRADE, MAY BE UTILIZED, CONTRACTORS PREFERENCE. CONTRACTOR SHALL COORDINATE WITH OWNER.
2. REEFER TO BEARING PAD & LATERAL SLIDE PLATE PLAN FOR ADDITIONAL INFORMATION.
3. UNLESS NOTED OTHERWISE, SUBGRADE SHALL UNDISTURBED SOIL OR COMPACTED STRUCTURAL FILL.
4. FIELD VERIFY ALLOWABLE BEARING CAPACITY. CONTRACTOR SHALL PROMPTLY REPORT BEARING CAPACITIES LESS THAN STATED IN GENERAL FOUNDATION NOTES BEFORE PROCEEDING WITH ADDITIONAL WORK.
   a. PROVIDE WASHED GRAVEL BELOW SLAB ON GRADE FOUNDATION TO SUBGRADE. GRAVEL SHALL EXTEND TO FROST DEPTH, MINIMUM 8" THICK.
5. TOP OF FOUNDATION ELEVATION TOLERANCE 1/4" IN 10'-0" & 1/2" MAX. OVERALL.
   a. SHIM W/ 4x4 NEOPRENE BEARING PADS TO ATTAIN SAME ELEVATION WITHIN 1/16"(±). SEE DRAWING S1.6 FOR ADDITIONAL INFORMATION.
6. FOUNDATION WIDTH AND LENGTH SHALL NOT EXCEED DIMENSIONS SHOWN.
   a. FOUNDATION WIDTH AND LENGTH SHALL NOT BE LESS THAN 1/4" LESS THAN DIMENSIONS SHOWN.
7. PROVIDE 12 MIL VAPOR BARRIER WITH 6" LAPPED & TAPED JOINTS BETWEEN SUBGRADE & SLAB ON GRADE FOUNDATION.

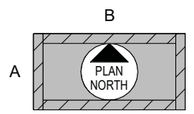
1 FOUNDATION PLAN (SLAB ON GRADE)
3/4" = 1'-0"

REBAR SCHEDULE			
ITEM #	TYPE	SHAPE	SPACING
9	#4	00	1' - 6"
10	#4	00	1' - 6"
1	#4	00	



2 SLAB ON GRADE FOUNDATION - SECTION
1" = 1'-0"

PARTS SCHEDULE - FOUNDATION	
ITEM #	DESCRIPTION
	ANCHOR, 1/2"-13 X 4-1/2" WEDGE ANCHOR STUD HDG HILTI
S1	BEARING PAD, 1/4" X 4" X 4" NEOPRENE
S2	LATERAL SLIDE PLATE, HDG



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Drawing Number:

Sheet Name
SLAB ON GRADE FOUNDATION PLAN & DETAILS

Project Number:

Sheet Number
S1.1B

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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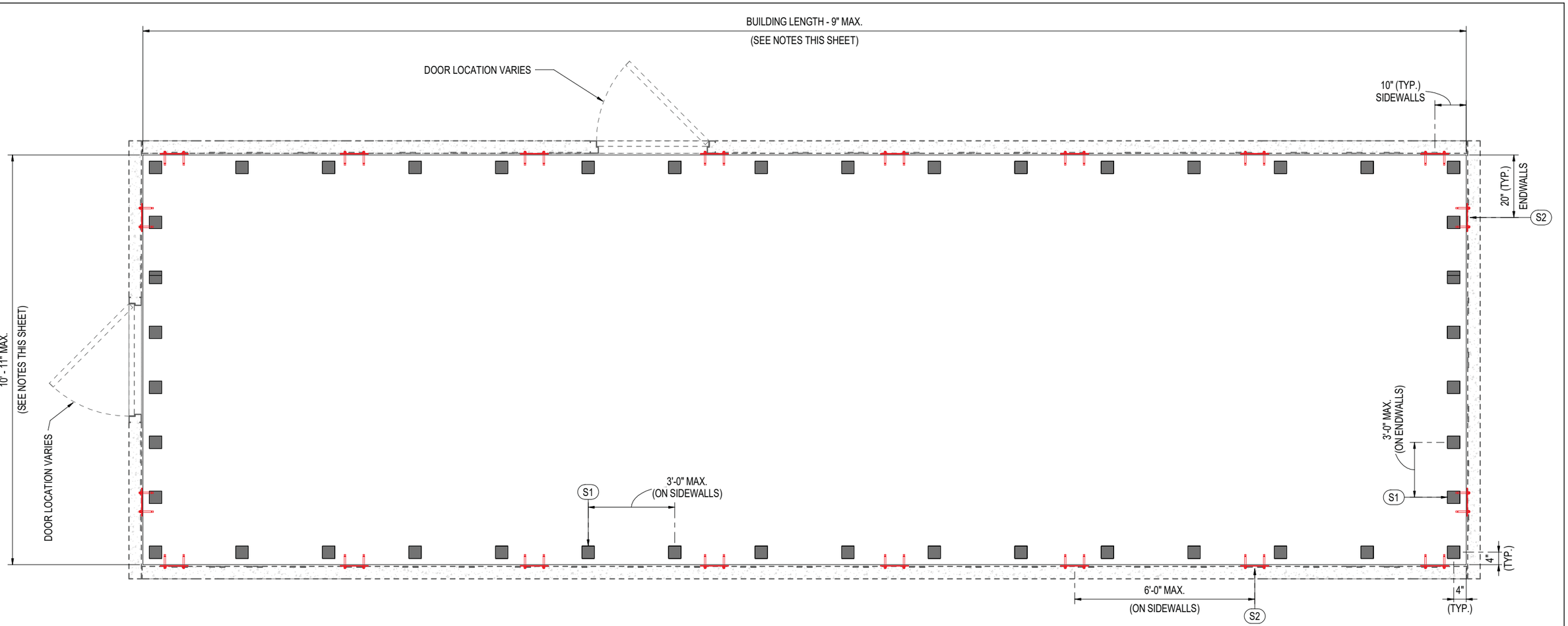
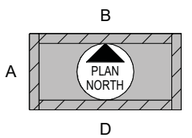


1 FOUNDATION PLAN (BEARING PADS)
3/4" = 1'-0"

Per Local Approval and Inspection

PARTS SCHEDULE - FOUNDATION

ITEM #	DESCRIPTION
	ANCHOR, 1/2"-13 X 4-1/2" WEDGE ANCHOR STUD HDG HILTI
S1	BEARING PAD, 1/4" X 4" X 4" NEOPRENE
S2	LATERAL SLIDE PLATE, HDG



KEYPLAN

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IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
FOUNDATION - BEARING PAD & SLIDE PLATE PLAN

Project Number:

Sheet Number
S1.6

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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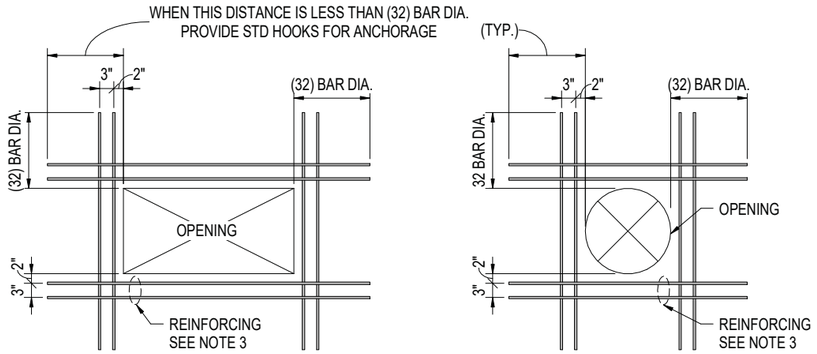
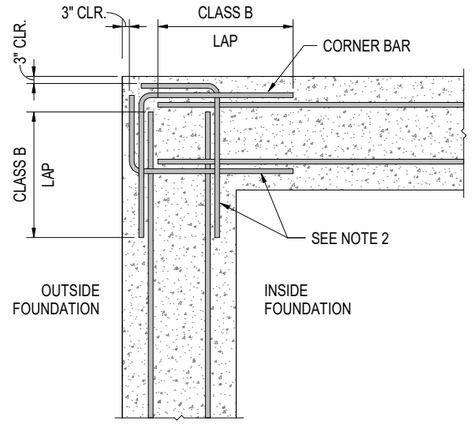


NOTES:

1. FOR OPENINGS WITH NO DIMENSIONS GREATER THAN 24": PROVIDE ADDITIONAL REINFORCEMENT AS SHOWN AND NOTED IN THESE DETAILS.
2. FOR OPENINGS WITH ANY DIMENSIONS GREATER THAN 24": PROVIDE ADDITIONAL REINFORCEMENT AS BILLED ON DRAWINGS. IF ADDITIONAL REINFORCEMENT NOT BILLED ON DRAWINGS, PROVIDE ADDITIONAL REINFORCEMENT AS SHOWN AND NOTED THESE DRAWINGS.
3. SIZE AND NUMBER OF LAYERS OF ADDITIONAL REINFORCEMENT TO BE THE SAME AS REINFORCEMENT IN SLAB OR GRADE BEAM ADJACENT TO THE OPENING. TYPICAL FOUR SIDES EXCEPT AS SHOWN OR NOTED ON DRAWINGS.

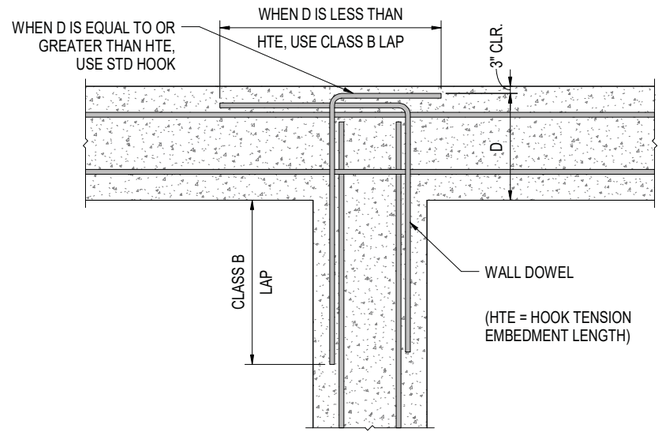
① TYPICAL REINFORCEMENT AT FOUNDATION OPENINGS FOR SLAB AND GRADE BEAMS
3/4" = 1'-0"



NOTES:

1. ALL CORNER BARS AND WALL DOWELS TO BE SAME SIZE AND SPACING AS HORIZONTAL WALL REINFORCEMENT UNLESS OTHERWISE SHOWN OR NOTED.
2. PROVIDE DOWELS WITH STANDARD HOOK AND CLASS B LAP.

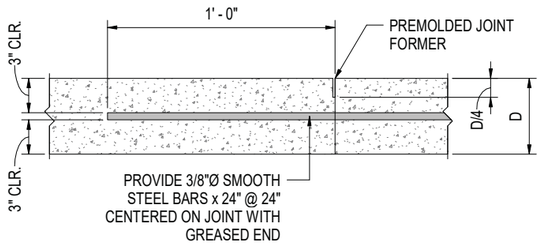
② S_TYPICAL CORNER DETAIL - WALLS AND GRADE BEAMS
1 1/2" = 1'-0"



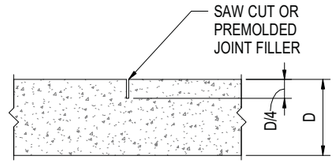
NOTES:

1. ALL CORNER BARS AND WALL DOWELS TO BE SAME SIZE AND SPACING AS HORIZONTAL WALL REINFORCEMENT UNLESS OTHERWISE SHOWN OR NOTED.
2. PROVIDE DOWELS WITH STANDARD HOOK AND CLASS B LAP.

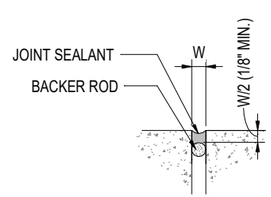
③ S_TYPICAL CORNER T DETAIL - WALLS AND GRADE BEAMS
1 1/2" = 1'-0"



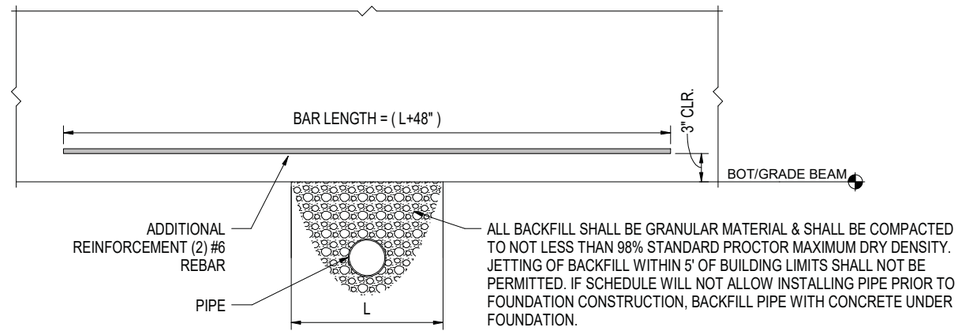
④ S_TYPICAL SLAB CONSTRUCTION JOINT
3" = 1'-0"



⑤ TYPICAL SLAB CONTROL JOINT
3" = 1'-0"



⑥ TYPICAL SEALANT AT SLAB JOINT
12" = 1'-0"



⑦ S_TYPICAL REINFORCEMENT AT PIPE BELOW FOOTING GRADE BEAM
1 1/2" = 1'-0"

NOTES:

1. DETAILS ARE TYPICAL AND MAY NOT ALL BE USED.

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Drawing Number:

Sheet Name
TYPICAL FOUNDATION DETAILS

Project Number:

Sheet Number
S1.7

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one quarter inch = one foot
0 4 8

three eighths inch = one foot
0 4

one half inch = one foot
0 4

three quarters inch = one foot
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one and one half inch = one foot
0 1

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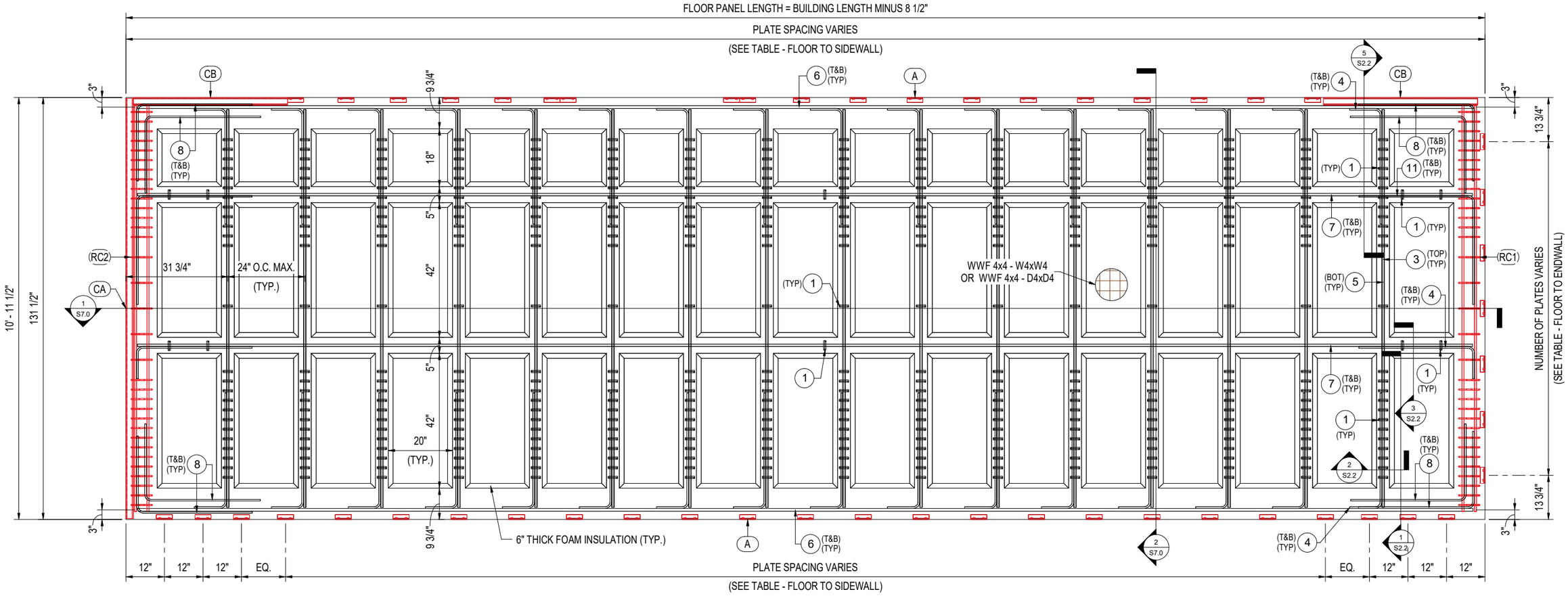


1 FLOOR PANEL
3/4" = 1'-0"

FLOOR CONNECTION PLATES

ROOF LIVE LOAD (PSF)	FLOOR TO SIDEWALL		FLOOR TO ENDWALL	
	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)	PLATE TYPE	# OF PLATES
100	PLATE 1	15"	PLATE 1	8
80	PLATE 1	18"	PLATE 1	7
35	PLATE 1	24"	PLATE 1	7

NOTE: C8x13.75 CHANNEL MAYBE SUBSTITUDED,
MININUM LENGTH 2'-0"
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF

PARTS SCHEDULE

ITEM #	DESCRIPTION
A	PLATE 1
CA	C8x13.75 CHANNEL PRIMED
CB	C8x13.75 CHANNEL PRIMED
RC1	FLOOR CAGE 12' WIDE (SEE DETAILS)
RC2	FLOOR CAGE 12' WIDE (SEE DETAILS)

REBAR

ITEM #	TYPE
1	#3
3	#4
4	#4
8	#4
11	#4
7	#6
5	#7
6	#7

4" RIB REINFORCEMENT (WIDTH)

BUILDING WIDTH	FLOOR LIVE LOAD	TOP REBAR	BOTTOM REBAR	STIRRUP SIZE	STIRRUP SPACING	# OF STIRRUPS (EACH END)
12'	250	#4	#7	#3	3"	12
	200	#4	#7	#3	3"	9
	150	#4	#7	#3	3"	4

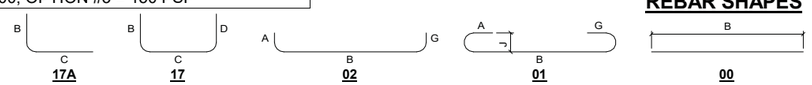
OPTION #1 = 250, OPTION #2 = 200, OPTION #3 = 150 PSF

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

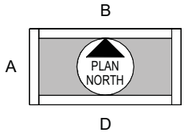
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

REBAR SHAPES



NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. FOR THE WALL PANELS, COORDINATE WITH MEP DRAWINGS FOR EXACT PENETRATION LOCATIONS.
3. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
4. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
5. FLOOR BLOCKOUTS ARE ONLY PERMISSIBLE ON (1) SIDE OF ANY GIVEN FLOOR RIB AND SHALL NOT VIOLATE THE RIB(S) OR BEAM(S) WIDTHS.
6. PENETRATIONS THROUGH WIDTH RIBS ARE NOT PERMISSIBLE.



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2024.05.29 15:27:53-04'00'

1090 Industrial Blvd.
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Ph# 205-980-4565
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205-655-9992

No. Revision Description — Date — By

Title: 12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site: IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name: 8" FLOOR PANEL

Project Number: — Sheet Number: S2.1

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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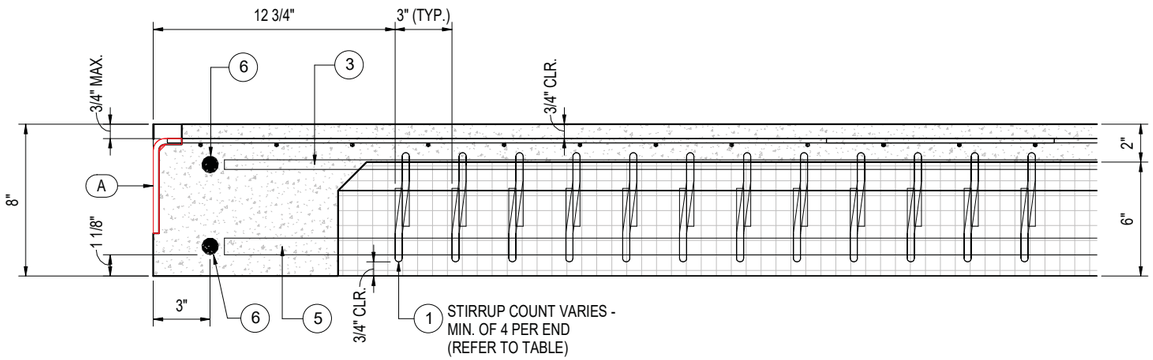
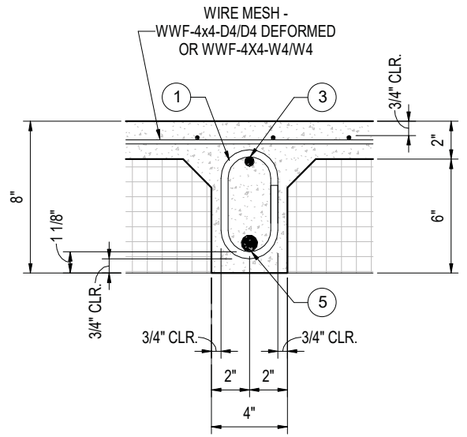
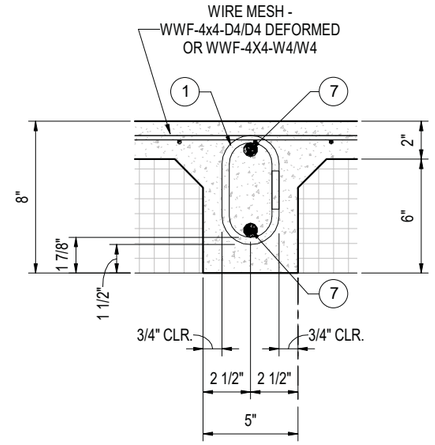


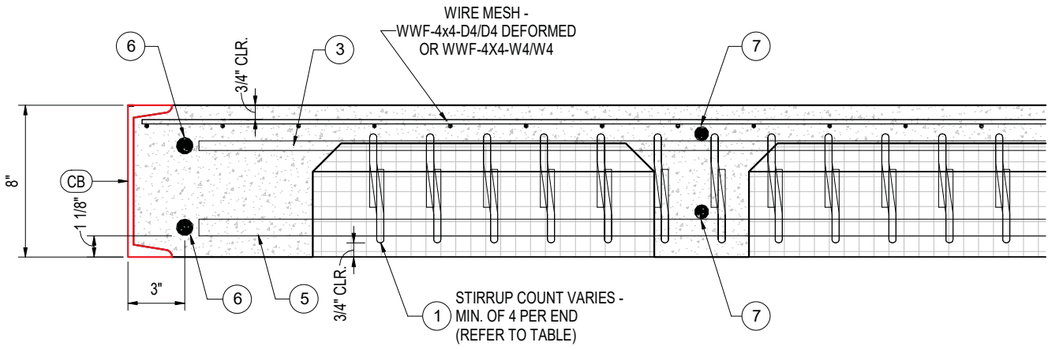
1 FLOOR STIRRUP DETAIL - EMBED PLATE OPTION
3" = 1'-0"



2 SECTION OF FLOOR @ WIDTH RIB
3" = 1'-0"



3 SECTION OF FLOOR @ LENGTH RIB
3" = 1'-0"



5 FLOOR STIRRUP DETAIL - CHANNEL OPTION
3" = 1'-0"

4" RIB REINFORCEMENT (WIDTH)

BUILDING WIDTH	FLOOR LIVE LOAD	TOP REBAR	BOTTOM REBAR	STIRRUP SIZE	STIRRUP SPACING	# OF STIRRUPS (EACH END)
12'	250	#4	#7	#3	3"	12
	200	#4	#7	#3	3"	9
	150	#4	#7	#3	3"	4
OPTION #1 = 250, OPTION #2 = 200, OPTION #3 = 150 PSF						

PARTS SCHEDULE

ITEM #	DESCRIPTION
A	PLATE 1
CB	C8x13.75 CHANNEL PRIMED

REBAR

ITEM #	TYPE
1	#3
3	#4
4	#4
8	#4
7	#6
5	#7
6	#7

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
3. FLOOR BLOCKOUTS ARE ONLY PERMISSIBLE ON (1) SIDE OF ANY GIVEN FLOOR RIB AND SHALL NOT VIOLATE THE RIB(S) OR BEAM(S) WIDTHS.
4. PENETRATIONS THROUGH WIDTH RIBS ARE NOT PERMISSIBLE.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER
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No.	Revision Description	Date	By

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Title:
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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
8" FLOOR PANEL - SECTIONS

Project Number:

Sheet Number
S2.2

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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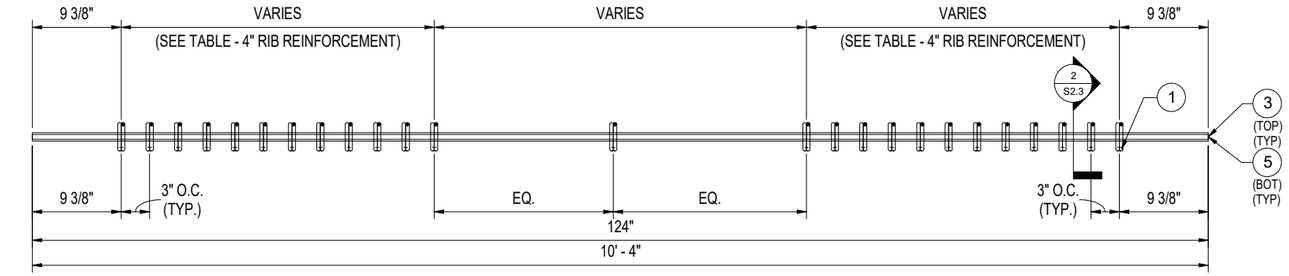
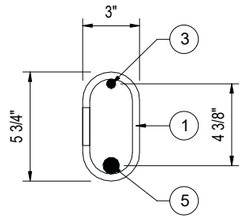
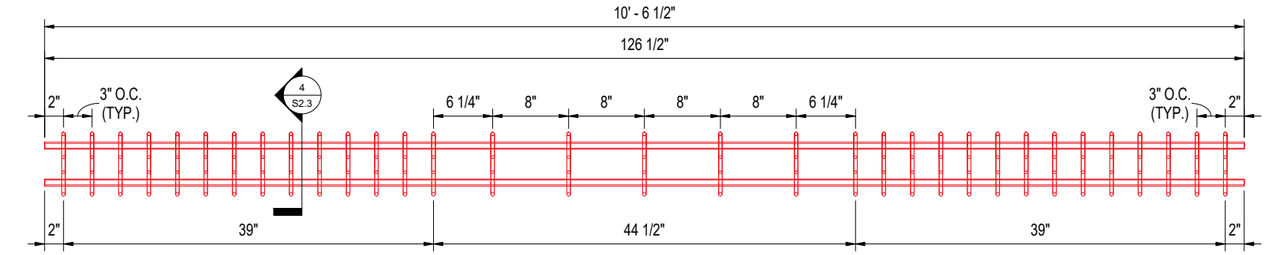


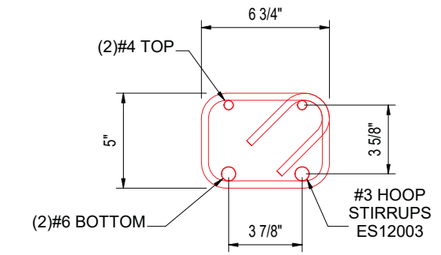
1 FLOOR PANEL - FAB - (FLOOR JOIST CAGE - PLAN VIEW)
1 1/2" = 1'-0"



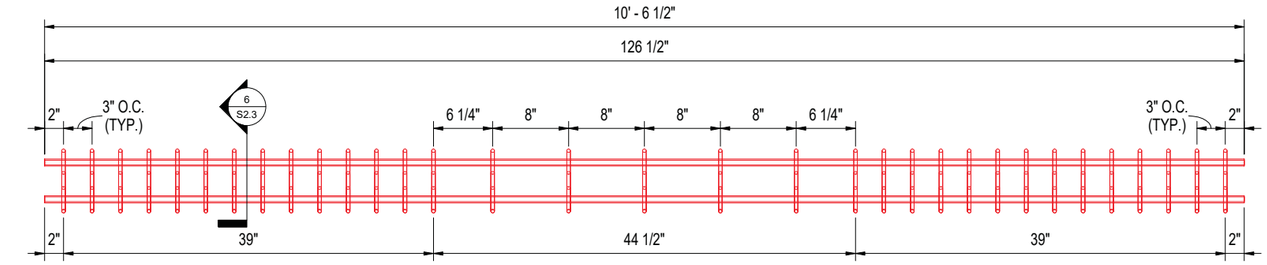
2 FLOOR PANEL - FAB - (FLOOR JOIST CAGE - SECTION VIEW)
3" = 1'-0"



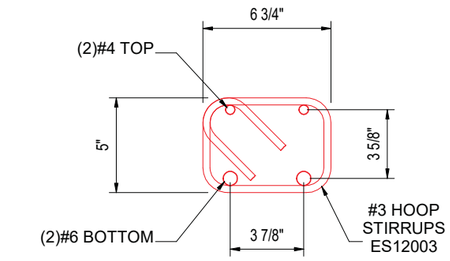
3 (RC2) FLOOR REBAR CAGE - PANEL C END (PLAN VIEW)
1 1/2" = 1'-0"



4 (RC2) FLOOR REBAR CAGE - PANEL C END (SECTION VIEW)
3" = 1'-0"



5 (RC1) FLOOR REBAR CAGE - PANEL A END (PLAN VIEW)
1 1/2" = 1'-0"



6 (RC1) FLOOR REBAR CAGE - PANEL A END (SECTION VIEW)
3" = 1'-0"

4" RIB REINFORCEMENT (WIDTH)						
BUILDING WIDTH	FLOOR LIVE LOAD	TOP REBAR	BOTTOM REBAR	STIRRUP SIZE	STIRRUP SPACING	# OF STIRRUPS (EACH END)
12'	250	#4	#7	#3	3"	12
	200	#4	#7	#3	3"	9
	150	#4	#7	#3	3"	4
OPTION #1 = 250, OPTION #2 = 200, OPTION #3 = 150 PSF						

PARTS SCHEDULE	
ITEM #	DESCRIPTION
RC1	FLOOR CAGE 12' WIDE (SEE DETAILS)
RC2	FLOOR CAGE 12' WIDE (SEE DETAILS)

REBAR	
ITEM #	TYPE
1	#3
3	#4
8	#4
5	#7

NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
3. FLOOR BLOCKOUTS ARE ONLY PERMISSIBLE ON (1) SIDE OF ANY GIVEN FLOOR RIB AND SHALL NOT VIOLATE THE RIB(S) OR BEAM(S) WIDTHS.
4. PENETRATIONS THROUGH WIDTH RIBS ARE NOT PERMISSIBLE.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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No. | Revision Description | Date | By

Title: 12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site: IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name: 8" FLOOR PANEL - REBAR CAGE & JOIST DETAILS

Project Number:

Sheet Number: S2.3

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one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
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one and one half inch = one foot (0, 1)

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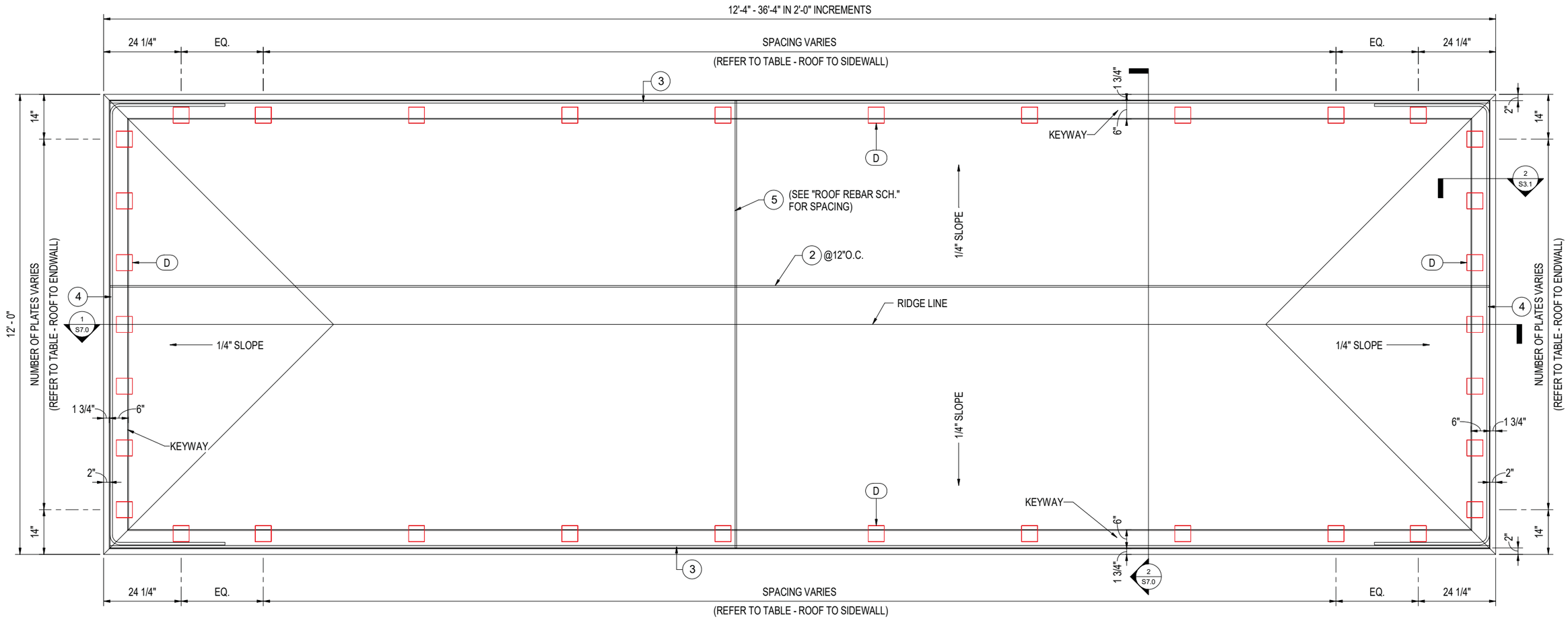


1 ROOF PANEL
3/4" = 1'-0"

ROOF CONNECTION PLATES

ROOF LIVE LOAD (PSF)	ROOF TO SIDEWALL		ROOF TO ENDWALL	
	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)	PLATE TYPE	# OF PLATES
100	PLATE 2	36"	PLATE 2	8
80	PLATE 2	48"	PLATE 2	7
35	PLATE 2	48"	PLATE 2	7

NOTE: MAX. SPACING ARE IN INCHES
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF

ROOF REBAR SCH.

ROOF LIVE LOAD (PSF)	REBAR SIZE	REBAR SPACING
100	#4	10"
80	#4	12"
35	#4	12"

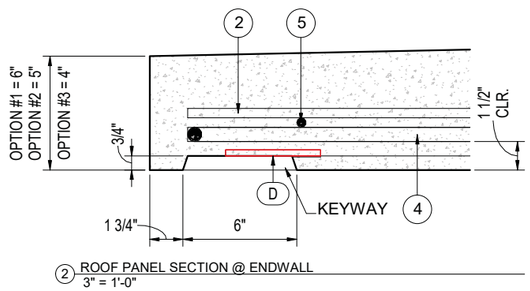
OPTION #1 = 100 PSF
OPTION #2 = 80 PSF
OPTION #3 = 35 PSF

PARTS SCHEDULE

ITEM #	DESCRIPTION
D	PLATE 2

REBAR

ITEM #	TYPE
2	#4
5	#4
3	#6
4	#6



2 ROOF PANEL SECTION @ ENDWALL
3" = 1'-0"

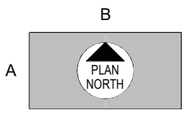
These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. FOR THE WALL PANELS, COORDINATE WITH MEP DRAWINGS FOR EXACT PENETRATION LOCATIONS.
3. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
4. CORNER BARS SHALL BE PRESENT @ ALL CORNERS. SIZE SAME AS PERIMETER BAR STANDARD LAP LENGTH PER DRAWING S0.1.
5. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.



KEYPLAN

REBAR SHAPES

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Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
6", 5", & 4" ROOF PANEL(S)

Project Number:

Sheet Number
S3.1

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three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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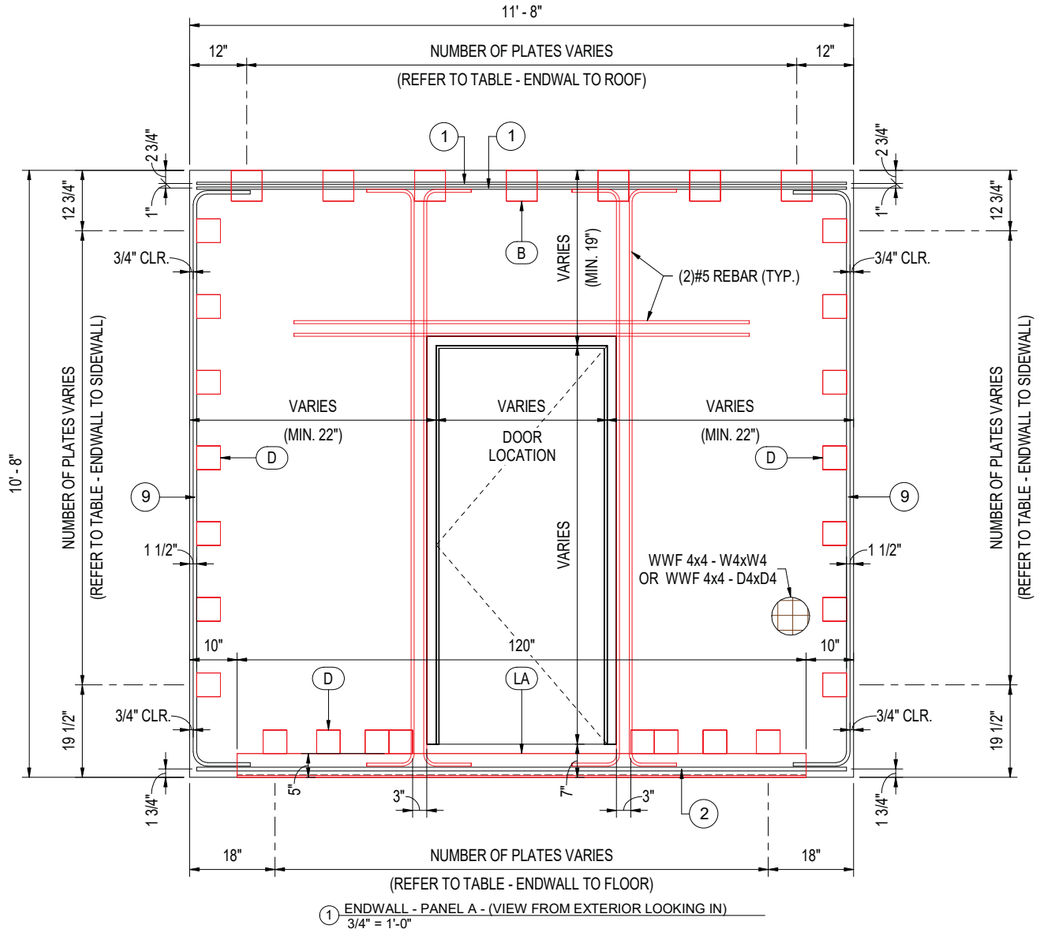

① ENDWALL - PANEL A - (VIEW FROM EXTERIOR LOOKING IN)
3/4" = 1'-0"

ENDWALL CONNECTION PLATES						
ROOF LIVE LOAD (PSF)	ENDWALL TO ROOF		ENDWALL TO SIDEWALL		ENDWALL TO FLOOR	
	PLATE TYPE	# OF PLATES	PLATE TYPE	# OF PLATES	PLATE TYPE	# OF PLATES
100	PLATE 49	8	PLATE 2	8	PLATE 2	8
80	PLATE 49	7	PLATE 2	7	PLATE 2	7
35	PLATE 49	7	PLATE 2	5	PLATE 2	7
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF						

PARTS SCHEDULE	
ITEM #	DESCRIPTION
B	PLATE 49
D	PLATE 2
D1	DOOR FRAME,
LA	3x5x3/8 THK ANGLE - PRIME PAINTED (WITH DOOR, CENTERED BELOW)

REBAR	
ITEM #	TYPE
1	#4
2	#6
9	#6

REBAR SHAPES

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

NOTES:
1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
3. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
4. THE TOTAL WIDTH OF OPENINGS ALONG ANY HORIZONTAL PLANE SHALL NOT EXCEED THE TOTAL NET WIDTH OF CONCRETE.
5. DOOR(S) AND OPENING(S) MAY BE USED IN ALL WALL PANELS.
6. DOOR(S) AND OPENING(S) SHALL COMPLY WITH OPENING REQUIREMENTS PER SHEET S0.1.
7. REFER TO CENTER-OF-GRAVITY PLAN DRAWINGS FOR ADDITIONAL LIFTING PLATE REQUIREMENTS.

A — PLAN NORTH

KEYPLAN

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LONNIE A MARVEL LICENSE No. 83750 STATE OF FLORIDA PROFESSIONAL ENGINEER
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Site: IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name: 4" ENDWALL PANEL A

Project Number: Sheet Number: S4.1

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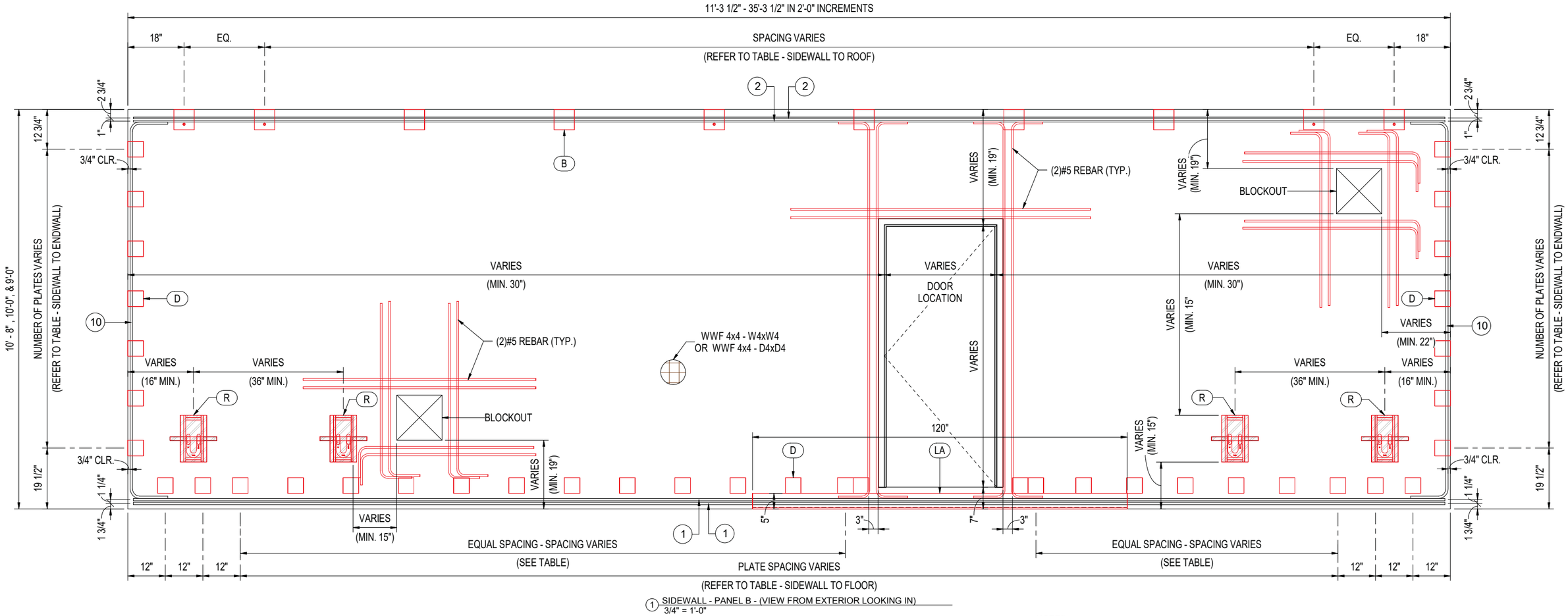

1 SIDEWALL - PANEL B - (VIEW FROM EXTERIOR LOOKING IN)
3/4" = 1'-0"

SIDEWALL CONNECTION PLATES

ROOF LIVE LOAD (PSF)	SIDEWALL TO ROOF		SIDEWALL TO ENDWALL		SIDEWALL TO FLOOR	
	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)	PLATE TYPE	# OF PLATES	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)
100	PLATE 49	36"	PLATE 2	8	PLATE 2	15"
80	PLATE 49	48"	PLATE 2	7	PLATE 2	18"
35	PLATE 49	48"	PLATE 2	5	PLATE 2	24"

NOTE: MAX. SPACING ARE IN INCHES
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF

PARTS SCHEDULE

ITEM #	DESCRIPTION
B	PLATE 49
C	PLATE 49A
D	PLATE 2
D2	DOOR FRAME,
LA	3x5x3/8 THK ANGLE - PRIME PAINTED (WITH DOOR, CENTERED BELOW)
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)

REBAR

ITEM #	TYPE
2	#4
1	#6
10	#6

NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
3. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
4. THE TOTAL WIDTH OF OPENINGS ALONG ANY HORIZONTAL PLANE SHALL NOT EXCEED THE TOTAL NET WIDTH OF CONCRETE.
5. DOOR(S) AND OPENING(S) MAY BE USED IN ALL WALL PANELS.
6. DOOR(S) AND OPENING(S) SHALL COMPLY WITH OPENING REQUIREMENTS PER SHEET S0.1.
7. REFER TO CENTER-OF-GRAVITY PLAN DRAWINGS FOR ADDITIONAL LIFTING PLATE REQUIREMENTS.

B
PLAN NORTH

KEYPLAN

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/260 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

REBAR SHAPES

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12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
4" SIDEWALL PANEL B

Project Number: Sheet Number S4.2

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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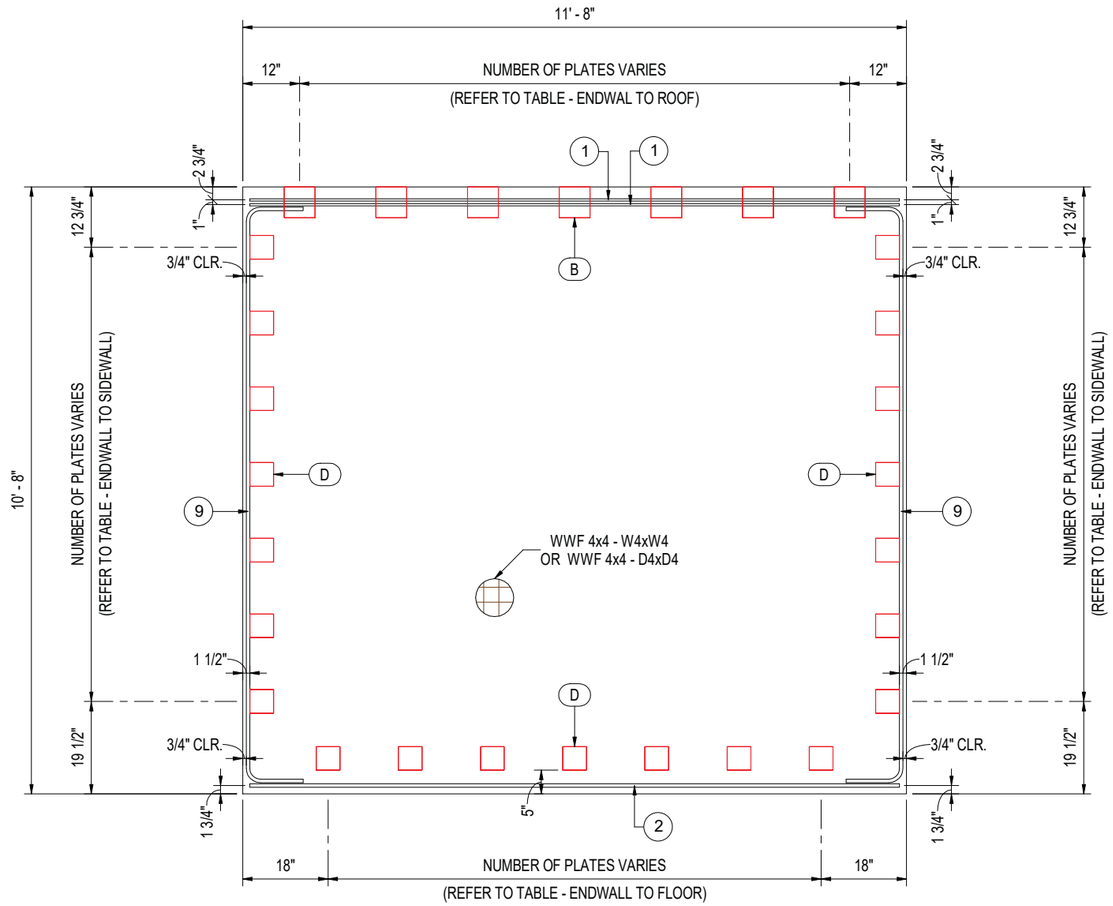


1 ENDWALL - PANEL C - (VIEW FROM EXTERIOR LOOKING IN)
3/4" = 1'-0"

ENDWALL CONNECTION PLATES

ROOF LIVE LOAD (PSF)	ENDWALL TO ROOF		ENDWALL TO SIDEWALL		ENDWALL TO FLOOR	
	PLATE TYPE	# OF PLATES	PLATE TYPE	# OF PLATES	PLATE TYPE	# OF PLATES
100	PLATE 49	8	PLATE 2	8	PLATE 2	8
80	PLATE 49	7	PLATE 2	7	PLATE 2	7
35	PLATE 49	7	PLATE 2	5	PLATE 2	7
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF						

PARTS SCHEDULE

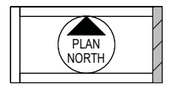
ITEM #	DESCRIPTION
B	PLATE 49
D	PLATE 2

REBAR

ITEM #	TYPE
1	#4
2	#6
9	#6

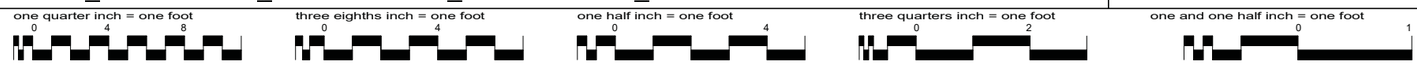
NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
3. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
4. THE TOTAL WIDTH OF OPENINGS ALONG ANY HORIZONTAL PLANE SHALL NOT EXCEED THE TOTAL NET WIDTH OF CONCRETE.
5. DOOR(S) AND OPENING(S) MAY BE USED IN ALL WALL PANELS.
6. DOOR(S) AND OPENING(S) SHALL COMPLY WITH OPENING REQUIREMENTS PER SHEET S0.1.
7. REFER TO CENTER-OF-GRAVITY PLAN DRAWINGS FOR ADDITIONAL LIFTING PLATE REQUIREMENTS.



KEYPLAN

REBAR SHAPES



These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116S MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LONNIE A MARVEL LICENSE No. 83750 STATE OF FLORIDA PROFESSIONAL ENGINEER
2024.05.29 15:28:26-04'00'

No.	Revision Description	Date	By

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1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.com
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Design & Inspection Services
www.tpengineeringinc.com
205-655-9992

Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
4" ENDWALL PANEL C

Project Number: Sheet Number S4.3

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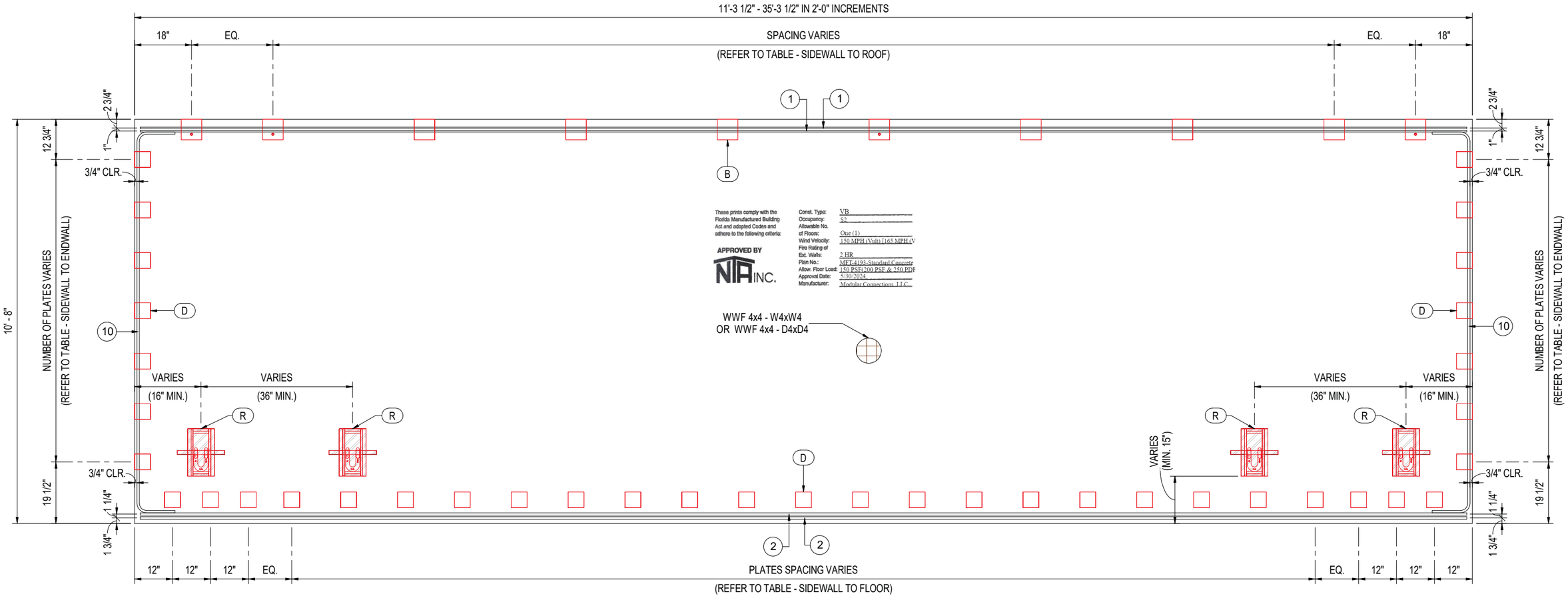


These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

WWF 4x4 - W4xW4
OR WWF 4x4 - D4xD4

1 SIDEWALL - PANEL D - (VIEW FROM EXTERIOR LOOKING IN)
3/4" = 1'-0"

SIDEWALL CONNECTION PLATES

ROOF LIVE LOAD (PSF)	SIDEWALL TO ROOF		SIDEWALL TO ENDWALL		SIDEWALL TO FLOOR	
	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)	PLATE TYPE	# OF PLATES	PLATE TYPE	MAX. O.C. SPACING (MIN. 4 PLATES)
100	PLATE 49	36"	PLATE 2	8	PLATE 2	15"
80	PLATE 49	48"	PLATE 2	7	PLATE 2	18"
35	PLATE 49	48"	PLATE 2	5	PLATE 2	24"

NOTE: MAX. SPACING ARE IN INCHES
OPTION #1 = 100, OPTION #2 = 80, OPTION #3 = 35 PSF

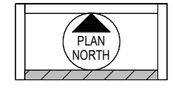
PARTS SCHEDULE

ITEM #	DESCRIPTION
B	PLATE 49
C	PLATE 49A
D	PLATE 2
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)

REBAR

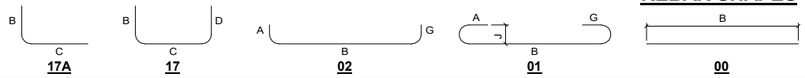
ITEM #	TYPE
1	#4
2	#6
10	#6

NOTES:
1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. REFER TO MISCELLANEOUS DETAILS FOR ADDITIONAL PLATE AND LIFT INFORMATION.
3. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.
4. THE TOTAL WIDTH OF OPENINGS ALONG ANY HORIZONTAL PLANE SHALL NOT EXCEED THE TOTAL NET WIDTH OF CONCRETE.
5. DOOR(S) AND OPENING(S) MAY BE USED IN ALL WALL PANELS.
6. DOOR(S) AND OPENING(S) SHALL COMPLY WITH OPENING REQUIREMENTS PER SHEET S0.1.
7. REFER TO CENTER-OF-GRAVITY PLAN DRAWINGS FOR ADDITIONAL LIFTING PLATE REQUIREMENTS.



D
KEYPLAN

REBAR SHAPES



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LICENSE
No. 83750
STATE OF FLORIDA
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No	Revision Description	Date	By

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Title:
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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
4" SIDEWALL PANEL D

Project Number:

Sheet Number
S4.4

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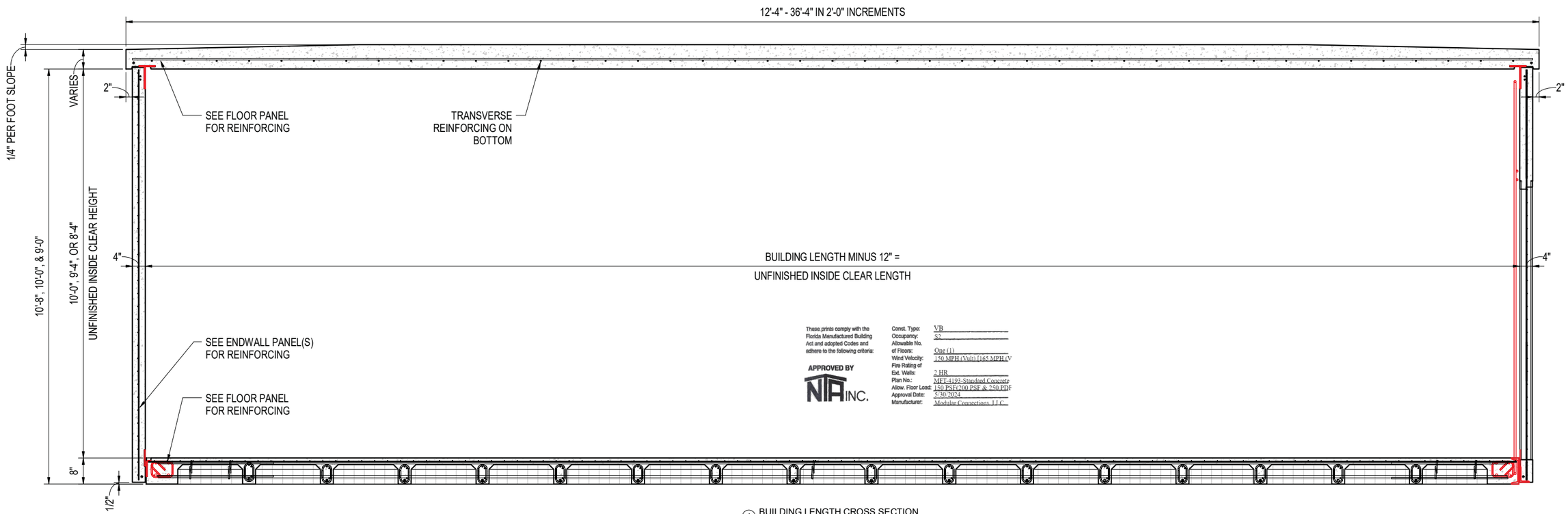
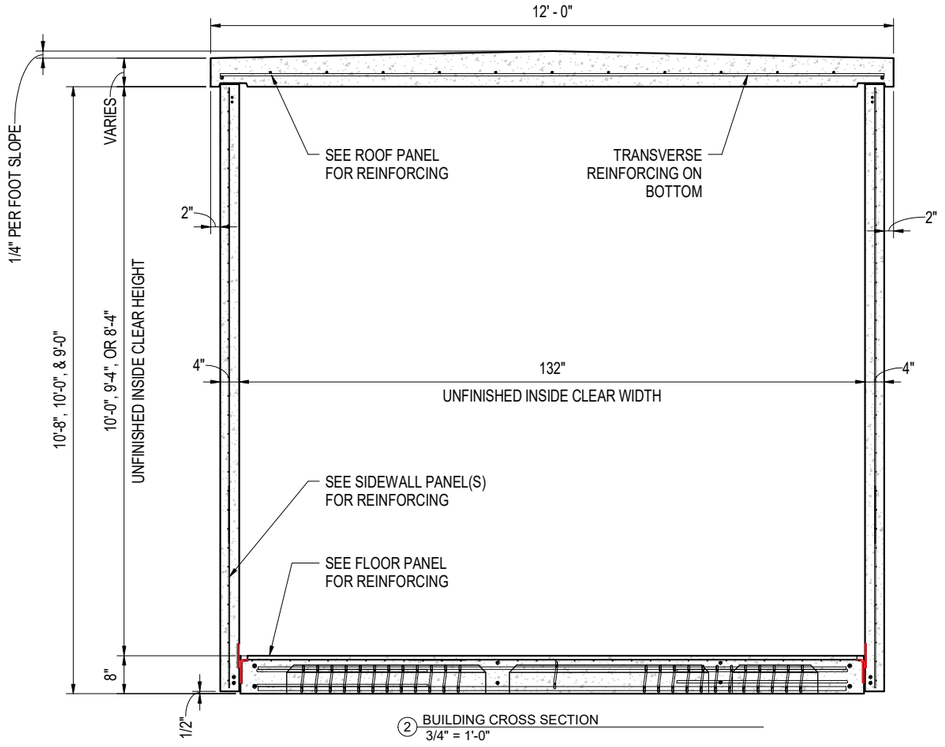


1 BUILDING LENGTH CROSS SECTION
3/4" = 1'-0"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



2 BUILDING CROSS SECTION
3/4" = 1'-0"

NOTES:

1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
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Title:
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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
BUILDING SECTIONS

Project Number:

Sheet Number
S7.0

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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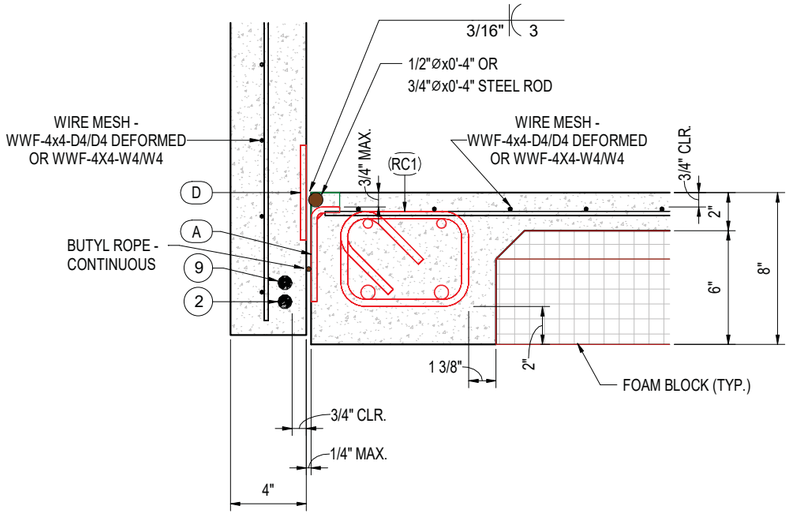
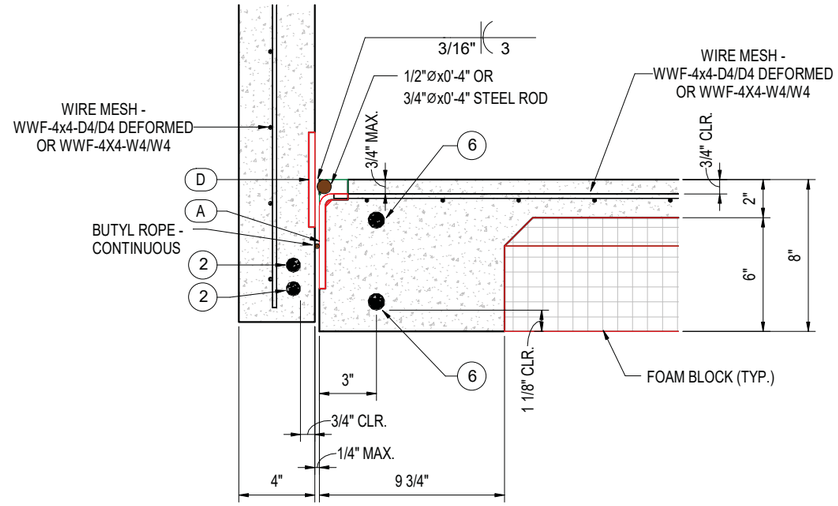


1 SECTION @ FLOOR TO END WALL CONNECTION - PLATE OPTION
3" = 1'-0"



2 SECTION @ FLOOR TO SIDE WALL CONNECTION - PLATE OPTION
3" = 1'-0"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

PARTS SCHEDULE

ITEM #	DESCRIPTION
A	PLATE 1
D	PLATE 2
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)
RC1	FLOOR CAGE 12' WIDE (SEE DETAILS)

REBAR

ITEM #	TYPE
2	#6
9	#6
6	#7

NOTES:
1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.

REBAR SHAPES

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LICENSE
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No	Revision Description	Date	By

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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
8" FLOOR CONNECTION DETAILS

Project Number:

Sheet Number
S8.0

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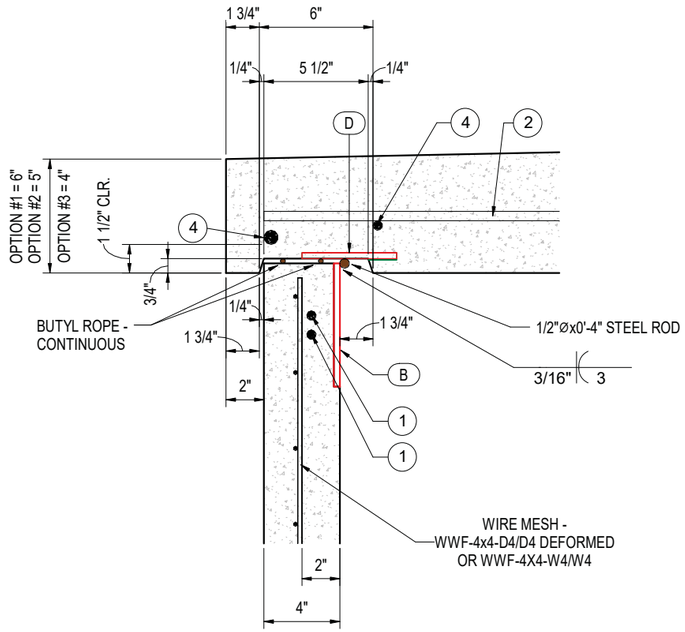
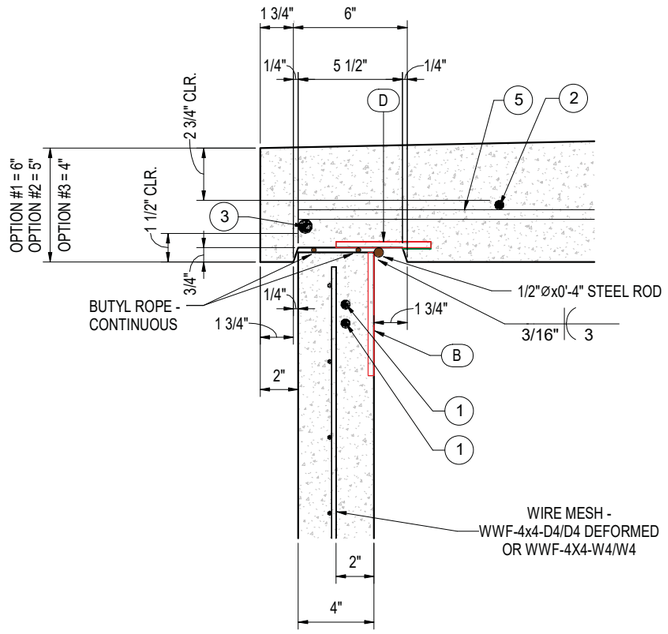


1 SECTION @ ROOF TO END WALL CONNECTION
3" = 1'-0"



2 SECTION @ ROOF TO SIDE WALL CONNECTION
3" = 1'-0"

PARTS SCHEDULE	
ITEM #	DESCRIPTION
B	PLATE 49
D	PLATE 2

REBAR	
ITEM #	TYPE
1	#4
2	#4
5	#4
3	#6
4	#6

NOTES:
1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
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Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
6", 5", & 4" ROOF CONNECTION DETAILS

Project Number:

Sheet Number
S8.1

one quarter inch = one foot
0 4 8

three eighths inch = one foot
0 4

one half inch = one foot
0 4

three quarters inch = one foot
0 2

one and one half inch = one foot
0 1

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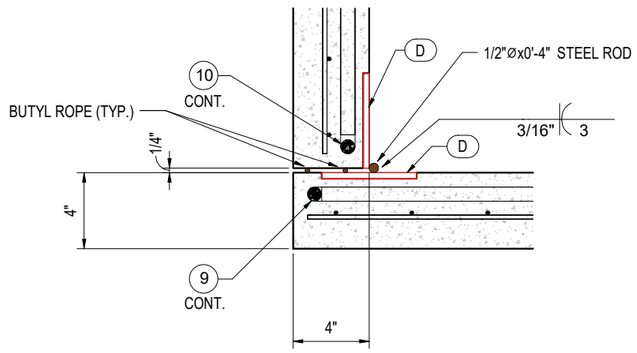
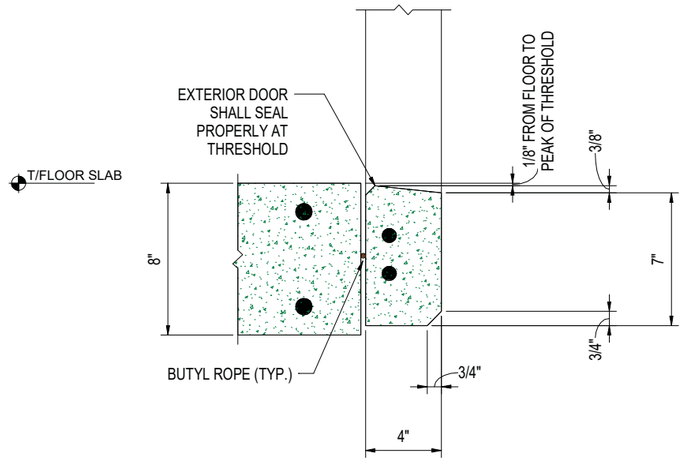


1 SECTION @ CORNER CONNECTION
3" = 1'-0"



2 DOOR THRESHOLD DETAIL
3" = 1'-0"

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

PARTS SCHEDULE	
ITEM #	DESCRIPTION
D	PLATE 2
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)

REBAR	
ITEM #	TYPE
2	#6
9	#6
10	#6

NOTES:
1. FOR STANDARD HOOKS AND LAP SPLICES, SEE SHEET S0.1.
2. 3/4" MINIMUM REBAR COVER UNLESS NOTED OTHERWISE ON DRAWING.

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LICENSE
No. 83750
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Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
4" WALL TO WALL CONNECTION & MISC. DETAILS

Project Number:

Sheet Number
S8.2

3/18/2024 4:47:19 PM
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three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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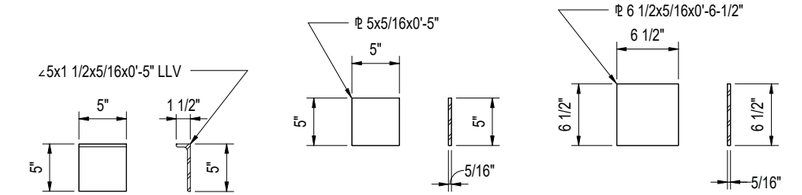
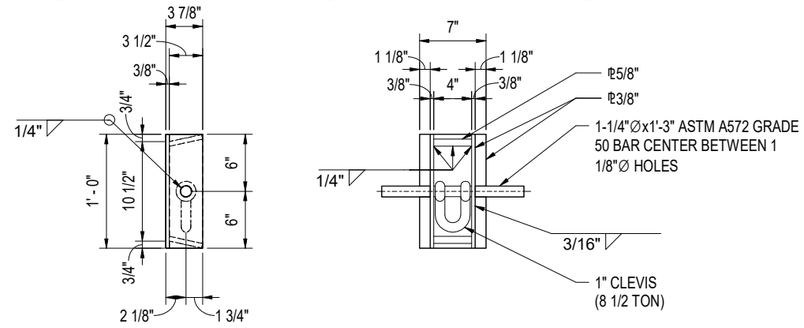


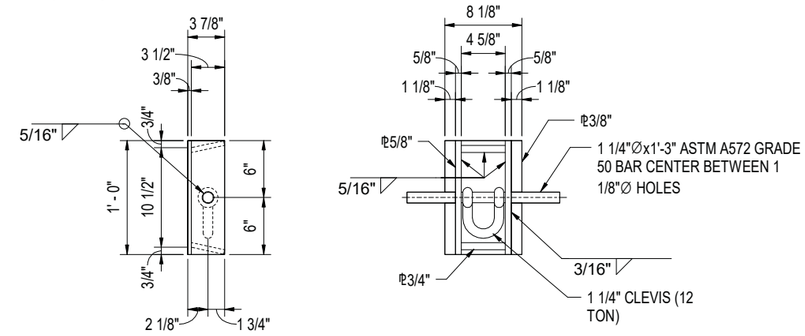
1 PLATE 1
1 1/2" = 1'-0"

2 PLATE 2
1 1/2" = 1'-0"

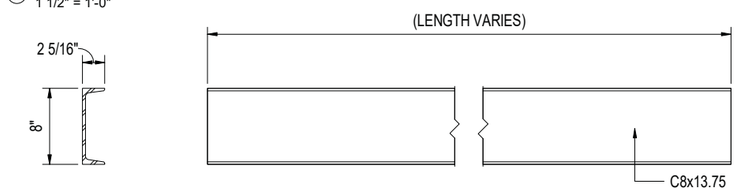
3 PLATE 49
1 1/2" = 1'-0"



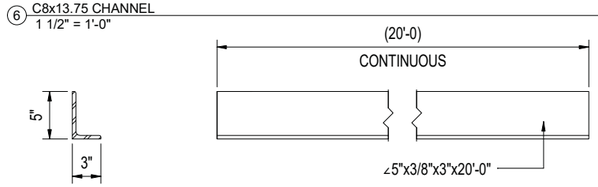
4 LIFTING PLATE - 8 1/2 TON
1 1/2" = 1'-0"



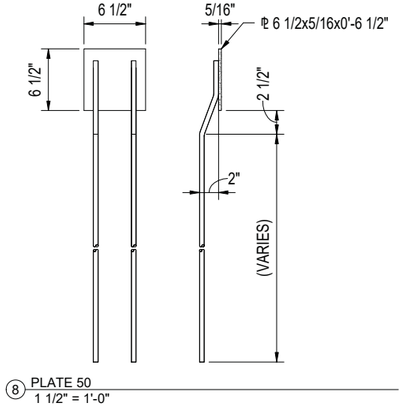
5 LIFTING PLATE - 10 TON
1 1/2" = 1'-0"



6 C8x13.75 CHANNEL
1 1/2" = 1'-0"



7 5x3x3/8 ANGLE
1 1/2" = 1'-0"



8 PLATE 50
1 1/2" = 1'-0"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFL-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER
2024.05.29 15:29:00-04'00'

No.	Revision Description	Date	By

MODULAR CONNECTIONS
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565
Email: info@modularconnections.com

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TP ENGINEERING, INC.
Design & Inspection Services
www.tpengineeringinc.com
205-655-9992

Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
MISCELLANEOUS DETAILS

Project Number:

Sheet Number
S9.0

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

NOTICE: These drawings and specifications are the property of Modular Connections, LLC. All information contained herein which is not known generally in the field of Modular Connections, LLC shall be confidential except to any extent to which it is established to have been known previously from sources other than Modular Connections, LLC. These drawings and specifications may not be reproduced, copied or used as the basis for the manufacture or sale of apparatus without written permission.

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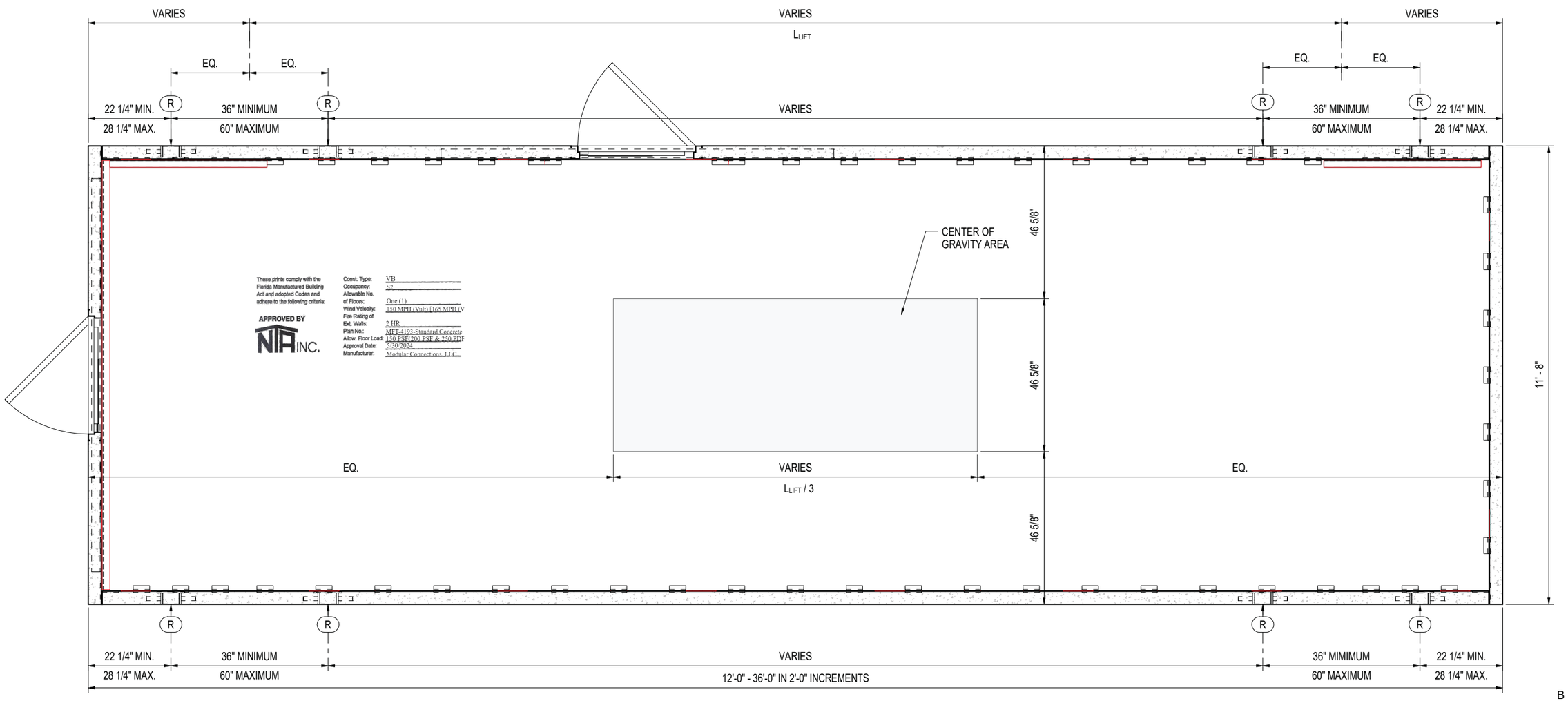

1 CENTER-OF-GRAVITY FLOOR PLAN - (8) LIFTS
3/4" = 1'-0"

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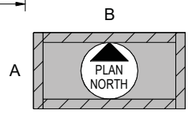
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LIFTING SCHEDULE

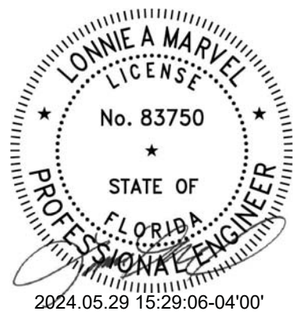
ITEM #	DESCRIPTION
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)
TOTAL WEIGHT	



KEYPLAN

NOTE:

1. LIFTING DEVICES SHOWN HAVE AN ALLOWABLE CAPACITY AS INDICATED,
2. MISCELLANEOUS EQUIPMENT, INTERIOR FINISHES, PACK-IN-SHELTER ITEMS, ETC. SHALL BE ARRANGED SUCH THAT THE CENTER OF GRAVITY OF THE MODULE SHALL BE WITHIN THE CENTER 1/3 AREA BETWEEN LIFTING DEVICES, AS INDICATED IN DETAIL 1/A1.0.
   A. UNDER NO CIRCUMSTANCES SHALL LIFTING OCCUR IF CENTER OF GRAVITY IS OUTSIDE OF ALLOWABLE AREA.
   B. UNDER NO CIRCUMSTANCES SHALL LIFTING OCCUR IF LOAD ARRANGEMENT CAUSES ANY LIFTING DEVICE TO EXCEED ITS STATED ALLOWABLE CAPACITY.
3. IT IS THE RESPONSIBILITY OF THE TRANSPORTATION AND CRANE CONTRACTOR TO ENSURE EQUIPMENT LOADS ARE DISTRIBUTED ACCORDINGLY.
4. LOCATE STATE DECAL AT LOAD DISTRIBUTION PANEL.



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No	Revision Description	Date	By

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1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.com

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Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
CENTER-OF-GRAVITY PLAN (SIDEWALL LIFT - 8)

Project Number: Sheet Number
S10.1

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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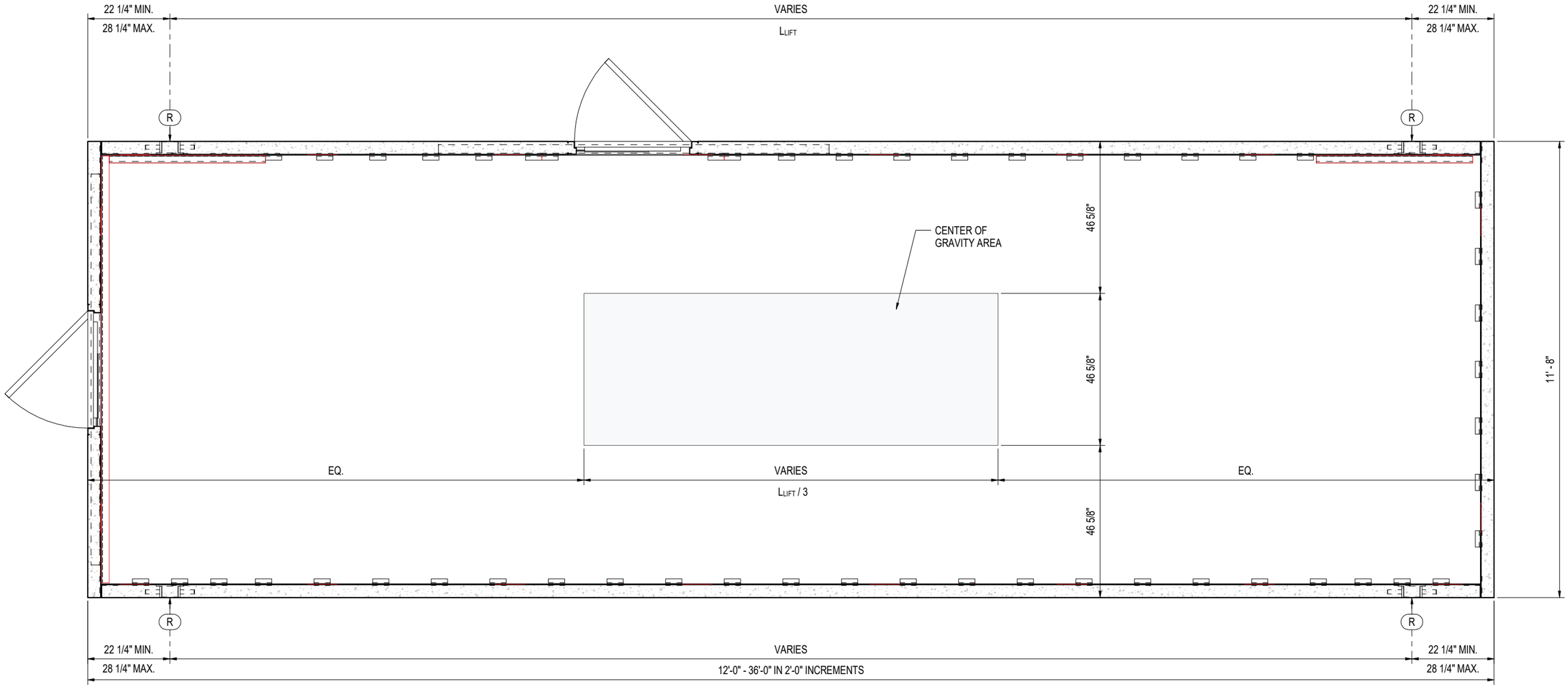
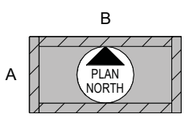


1 CENTER-OF-GRAVITY FLOOR PLAN - (4) LIFTS
3/4" = 1'-0"

LIFTING SCHEDULE	
ITEM #	DESCRIPTION
R	10-TON LIFTING PLATE HDG (OPTIONAL 8-TON)
TOTAL WEIGHT	



KEYPLAN

NOTE:

1. LIFTING DEVICES SHOWN HAVE AN ALLOWABLE CAPACITY AS INDICATED,
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3. IT IS THE RESPONSIBILITY OF THE TRANSPORTATION AND CRANE CONTRACTOR TO ENSURE EQUIPMENT LOADS ARE DISTRIBUTED ACCORDINGLY.
4. LOCATE STATE DECAL AT LOAD DISTRIBUTION PANEL.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
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No	Revision Description	Date	By

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Email: info@modularconnections.com
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Title:
12'W x 12'-36'L x 8'-0", 9'-0", OR 9'-8" NOMINAL I.H.

Site
IBC 2021

Drawn By:	Checked By:	Project Manager:	Date:
LMS	CLP	LAM	3/11/24

Drawing Number:

Sheet Name
CENTER-OF-GRAVITY PLAN (SIDEWALL LIFT - 4)

Project Number:

Sheet Number
S10.2

one quarter inch = one foot (0, 4, 8)
three eighths inch = one foot (0, 4)
one half inch = one foot (0, 4)
three quarters inch = one foot (0, 2)
one and one half inch = one foot (0, 1)

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MODULAR CONNECTIONS, LLC

Cover =

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

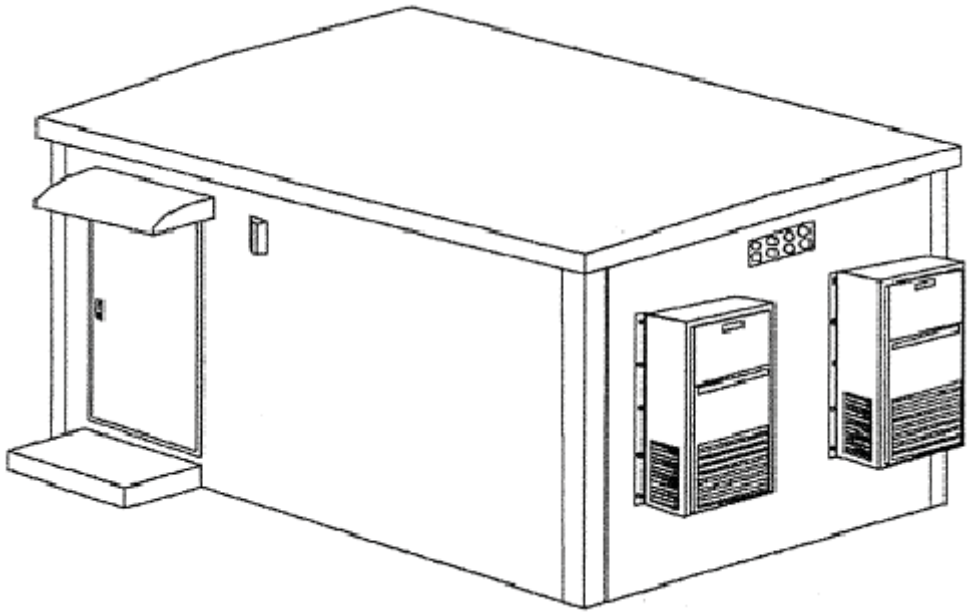
" IBC 2021 (ASCE 7-16) "

" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #1"

" REVISION - 0: DATE MARCH 11, 2024 "

""



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" copied or traced in whole or part without our written consent. "

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

TABLE OF CONTENTS

Pages =

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" ROOF DESIGN ---------- S2.1 - S2.6 "
" WALL DESIGN ---------- S3.1 - S3.16 "
" FLOOR DESIGN ---------- S4.1 - S4.31 "
" LATERAL DESIGN ---------- S5.1 - S5.22 "
" FOUNDATION DESIGN ---------- S6.1 - S6.8 "
" CONNECTION DESIGN ---------- S7.1 - S7.33 "
" TRANSPORTATION DESIGN ---------- S8.1 - S8.15 "
" FIRE RATING & THERMAL EFFICIENCY---------- S9.1 - S9.3 "
" WALL OPENING DESIGN ---------- S10.1 - S10.4 "
""
""
" APPENDIX A - BULLET RESISTANCE TEST "
" APPENDIX B - AWINING / HOOD ATTACHMENT DESIGN "
" APPENDIX C - DEVELOPMENT OF REINFORCEMENT "

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Allowable No. of Floors:	One (1)
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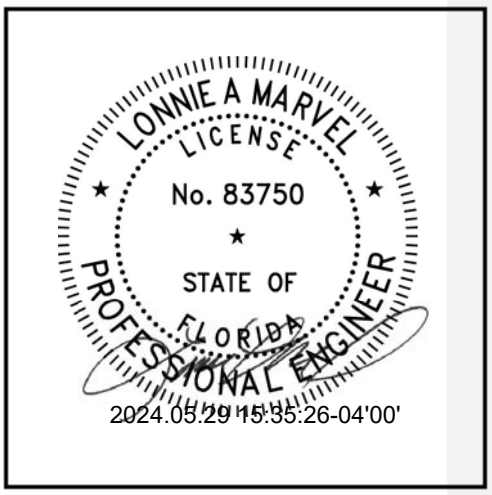
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.



MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

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Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 37.4 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 66.4 psf"
"Roof Live Load ---------- 100 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 150 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 35.8 psf"
"Uplift Wind Pressure, MWFRS ---------- 53.5 psf"
"Zone 4 Wind Pressure, C & C ---------- 53.3 psf"
"Zone 5 Wind Pressure, C & C ---------- 65.8 psf"
"Zone 1 Wind Pressure, C & C ---------- 78.2 psf"
"Zone 2 Wind Pressure, C & C ---------- 103.2 psf"
"Zone 3 Wind Pressure, C & C ---------- 140.7 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 32 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 39.5 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.58"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.14"

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$)
Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)
Welding electrodes : E70XX

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"
$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"
$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"
$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 100 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"
$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot psf \qquad q_h = 41.616 \cdot psf \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 20.81 & 35.79 \\ -25.18 & -10.2 \\ -32.25 & -17.27 \\ -53.48 & -38.49 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 35.79 & 20.81 \\ -10.2 & -25.18 \\ -17.27 & -32.25 \\ -38.49 & -53.48 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 35.79 \\ 25.18 \\ 32.25 \\ -53.48 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 45.99 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 41.616 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -65.8 & -50.8 \\ 5 & 20 \\ -78.2 & -63.3 \\ 5 & 20 \\ -103.2 & -88.2 \\ 5 & 20 \\ -140.7 & -125.7 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -50.8 & -65.8 \\ 20 & 5 \\ -63.3 & -78.2 \\ 20 & 5 \\ -88.2 & -103.2 \\ 20 & 5 \\ -125.7 & -140.7 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 53.3 \\ 53.3 \\ 65.8 \\ 78.2 \\ 103.2 \\ 140.7 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 65.8 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 78.2 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 103.2 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 140.7 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 107.369 \cdot psf$$

Max Roof Uplift Pressure (average)

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 0.7 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1.24 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 0.868$ $S_{MS.ref} =$ "Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.579$ $S_{DS.ref} =$ "5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} =$ "Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} =$ "5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"

$$H_E = \begin{pmatrix} 9.646 \\ 10.646 \\ 11.313 \end{pmatrix} \text{ft}$$

$H_{E.ref} =$ "Exterior Height of Building"

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right]$$

$Ta = 0.104$ $Ta_{ref} =$ "Approx Per., Eqn (12.8-7)"

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}} \right)$ $T_0 = 0.29$ $T_s := \left(\frac{S_{D1}}{S_{DS}} \right)$ $T_s = 1.449$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.1447$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E} \right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E} \right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E} \right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E} \right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E} \right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E} \right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1} \right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0255$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0 \right) \right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max} \right), C_{s.min} \right)$ $C_s = 0.145$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 6 \cdot in$ Thickness @ ridge $t_{r.r} = 7.5 \cdot in$ $t_{r.a} = 6.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.162 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 66.38 \cdot psf$ $DL_{r.mn.ref}$:= "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 73.38 \cdot psf$ $DL_{r.mx.ref}$:= "Max. Roof DL"

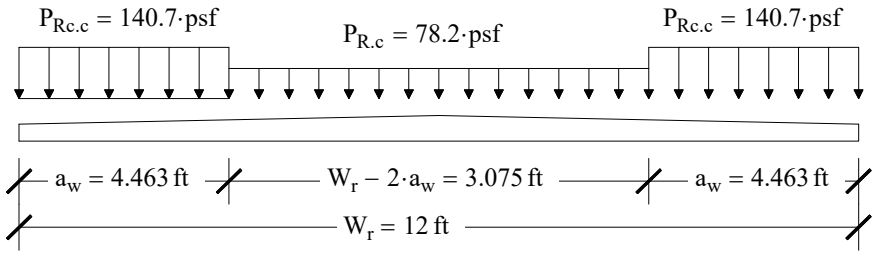
$w_{llr} = 100 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 140.7 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 78.2 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.463\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -24.358 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -112.769 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 8.492 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -8.492 \cdot psf$ Minimum Upward Roof Seismic Load

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

LRFD =	"1a"	"1.4(D+F)"	$Wu_{S2.0}$ =	102.72
	"2a"	"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "		138.05
	"3a"	"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"		248.05
	"3b+"	"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"		248.05
	"3b-"	"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"		191.67
	"4a+"	"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"		138.05
	"4a-"	"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"		81.67
	"5a+"	"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"		102.89
	"5a-"	"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"		85.91
	"6a+"	"0.9(D+F)+1.0(W+)+1.6(H)"		59.74
	"6a-"	"0.9(D+F)+1.0(W-)+1.6(H)"		–53.03
	"7a+"	"0.9(D+F)+1.0(E+)+1.6(H)"		68.23
	"7a-"	"0.9(D+F)+1.0(E-)+1.6(H)"		51.25

·psf Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref}$ = "Max Span of Roof Panel"

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 248.05 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -53.031 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

	$Mu_{p.ref}$ = "Ultimate Moment, +ve"	Vu_{ref} = "Ultimate Shear"
Roof Live	$Mu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}$	$Vu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}$
Load	$Mu_{2.1.mx} = 45.021 \cdot kip \cdot in$	$Vu_{2.1.mx} = 1.364\,kip$
Option 1	$Mu_{2.1.mn} := \min\left(0kip \cdot in, \lvert w_{u.mn} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$	$Vu_{2.1.mn} := \min\left(0kip, \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}\right)$
	$Mu_{2.1.mn} = 0 \cdot kip \cdot in$	$Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 100 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$B_{2.1}$ = "#4" $B_{2.1.ref}$ = "Transverse Rebar Size" $S_{2.1} = 10 \cdot in$ $S_{2.1.ref}$ = "Transverse Rebar Spacing"

$B_{2.2}$ = "#4" $B_{2.2.ref}$ = "Longitudinal Rebar Size" $S_{2.2} = 12 \cdot in$ $S_{2.2.ref}$ = "Longitudinal Rebar Spacing"

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref}$ = "Clear Cover, From Bot. w/ Keyway"

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref}$ = "Clear Cover, From Bot. w/o Keyway"

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref}$ = "Roof Width Bar Diam." $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref}$ = "Roof Width Bar Area."

$As_{2.1} = 0.24 \cdot in^2$ $As_{2.1.ref}$ = "Roof Width Area Reinf." $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref}$ = "Roof Length Bar Diam."

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref}$ = "Roof Length Bar Area." $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref}$ = "Roof Length Area Reinf."

$A_{s.mim.2} = 0.162 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
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$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.24 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.004$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.314 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 69.7 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.38 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.03645$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 62.8 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 45 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 3.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 4.345 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 17.106 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 4.345 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 3.258 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 1.364 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{F.2.3} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\quad \rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $\quad a_{2.5} = 0.314 \cdot in$ $\quad a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $\quad M_{n.2.3} = 22.9 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $\quad c_{2.3} = 0.38 \cdot in$ $\quad c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\quad \varepsilon_{2.3} = 0.01081$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\quad \phi M_{n.2.3} = 20.6 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $\quad Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.5} = \text{"OK"}$

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{V.2.4} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\quad \lambda_{s.2.4} = 1$ $\quad \lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$\rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.a} = 2.158 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\quad \phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $\quad Vc_{2.4} = 2.158 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\quad \phi Vc_{2.4} = 1.618 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Vu_{2.1.mn} = 0 \cdot kip$ $\quad Vu_{ref} = \text{"Ultimate Shear"}$

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.6} = \text{"OK"}$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 307.547 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 3.375 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 38.97 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 31.468 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 13.318 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 153.773 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -10.544 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 76.887 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 76.89 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 307.5 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 157.3 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.02628 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.12139 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.09511 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.269\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.062\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.331\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 398.6$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

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3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 100 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 100 \cdot psf$

$P_{W.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_0} =$ " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_1} =$ " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 65.8 \cdot psf$ $GC_{p.cc.max.ref_2} =$ "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 53.27 \\ 65.75 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 100 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.19 \\ 1.32 \\ 1.98 \\ 1.98 \\ 2.3 \\ 1.32 \\ 1.64 \\ 1.11 \\ 1.01 \\ 0.68 \\ 1.32 \\ 0.73 \\ 0.63 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 26.6 \\ 26.6 \\ 53.3 \\ 26.6 \\ 6.6 \\ -6.6 \\ 53.3 \\ 53.3 \\ 6.6 \\ -6.6 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))\ " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 2.3 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.54 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.63 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.07 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.47 \cdot kip$

$P_{u.3b.h1}^T = (1.188\ \ 1.318\ \ 1.978\ \ 1.978\ \ 2.3\ \ 1.318\ \ 1.64\ \ 1.107\ \ 1.005\ \ 0.677\ \ 1.321\ \ 0.728\ \ 0.626) \cdot kip$

$M_{u.3b.h1}^T = (0\ \ 0\ \ 0\ \ 0.27\ \ 0.27\ \ 0.539\ \ 0.27\ \ 0.066\ \ -0.066\ \ 0.539\ \ 0.539\ \ 0.066\ \ -0.066) \cdot kip \cdot ft$

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.12 \;\; 0.12 \;\; 0.24 \;\; 0.12 \;\; 0.03 \;\; -0.03 \;\; 0.24 \;\; 0.24 \;\; 0.03 \;\; -0.03)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ & Roof Live Load $w_{llr} = 100\cdot psf$

$Pu_{3b.h2}^{T} = (1.25 \;\; 1.37 \;\; 2.03 \;\; 2.03 \;\; 2.35 \;\; 1.37 \;\; 1.69 \;\; 1.16 \;\; 1.06 \;\; 0.71 \;\; 1.36 \;\; 0.76 \;\; 0.66)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 6.6 \;\; -6.6 \;\; 53.3 \;\; 53.3 \;\; 6.6 \;\; -6.6)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2}) \qquad P_{u.3b.h2.mx} = 2.35\cdot kip \qquad M_{u.3b.h2.mx} := \max(M_{u.3b.h2}) \qquad M_{u.3b.h2.mx} = 0.67\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2}) \qquad P_{u.3b.h2.mn} = 0.66\cdot kip \qquad M_{u.3b.h2.mn} := \min(M_{u.3b.h2}) \qquad M_{u.3b.h2.mn} = -0.08\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|) \qquad V_{u.3b.h2.mx} = 0.52\cdot kip$

$P_{u.3b.h2}^{T} = (1.251 \;\; 1.372 \;\; 2.032 \;\; 2.032 \;\; 2.354 \;\; 1.372 \;\; 1.694 \;\; 1.161 \;\; 1.059 \;\; 0.712 \;\; 1.357 \;\; 0.763 \;\; 0.661)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.333 \;\; 0.333 \;\; 0.666 \;\; 0.333 \;\; 0.082 \;\; -0.082 \;\; 0.666 \;\; 0.666 \;\; 0.082 \;\; -0.082)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.133 \;\; 0.133 \;\; 0.266 \;\; 0.133 \;\; 0.033 \;\; -0.033 \;\; 0.266 \;\; 0.266 \;\; 0.033 \;\; -0.033)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667\,ft$ & Roof Live Load $w_{llr} = 100\cdot psf$

$Pu_{3b.h3}^{T} = (1.29 \;\; 1.41 \;\; 2.07 \;\; 2.07 \;\; 2.39 \;\; 1.41 \;\; 1.73 \;\; 1.2 \;\; 1.1 \;\; 0.74 \;\; 1.38 \;\; 0.79 \;\; 0.69)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 6.6 \;\; -6.6 \;\; 53.3 \;\; 53.3 \;\; 6.6 \;\; -6.6)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3}) \qquad P_{u.3b.h3.mx} = 2.39\cdot kip \qquad M_{u.3b.h3.mx} := \max(M_{u.3b.h3}) \qquad M_{u.3b.h3.mx} = 0.76\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3}) \qquad P_{u.3b.h3.mn} = 0.69\cdot kip \qquad M_{u.3b.h3.mn} := \min(M_{u.3b.h3}) \qquad M_{u.3b.h3.mn} = -0.09\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|) \qquad V_{u.3b.h3.mx} = 0.55\cdot kip$

$P_{u.3b.h3}^{T} = (1.293 \;\; 1.409 \;\; 2.069 \;\; 2.069 \;\; 2.391 \;\; 1.409 \;\; 1.731 \;\; 1.198 \;\; 1.096 \;\; 0.736 \;\; 1.38 \;\; 0.787 \;\; 0.685)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.379 \;\; 0.379 \;\; 0.758 \;\; 0.379 \;\; 0.093 \;\; -0.093 \;\; 0.758 \;\; 0.758 \;\; 0.093 \;\; -0.093)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.142 \;\; 0.142 \;\; 0.284 \;\; 0.142 \;\; 0.035 \;\; -0.035 \;\; 0.284 \;\; 0.284 \;\; 0.035 \;\; -0.035)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\qquad S_{3.0}$ = "N/A" $\cdot in \qquad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \qquad d_{3b} := \frac{t_w}{2} - 0.25in \qquad d_{3b} = 1.75\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.555 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.087$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.92$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

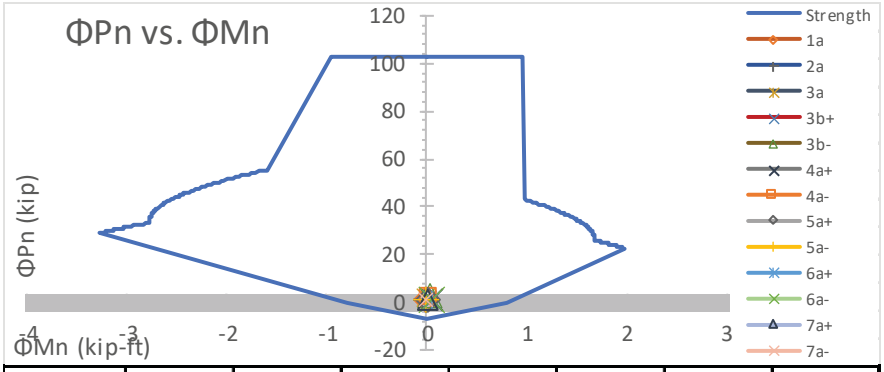
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.112$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.899$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.132$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.883$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.883$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.3%	1.4%	2.0%	2.94%	2.9%	6.1%	3.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.2%	1.1%	6.4%	6.1%	0.8%	0.8%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref}$ = "Min. Space to Adjacent Opening"

$b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref}$ = "Strip Width Btwn Openings"

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1}$ = "#6" $B_{3.1.ref}$ = "Wall Vert. Perim. Reinf." $B_{3.2}$ = "#4" $B_{3.2.ref}$ = "Wall Top. Perim. Reinf."

$N_{B.3.1} = 1$ $N_{B.3.1.ref}$ = "# Wall Vert. Perim. Bars" $N_{B.3.2} = 2$ $N_{B.3.2.ref}$ = "# Wall Top Perim. Bars"

$D_{3c.1} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.2} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.1} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.2} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.3}$ = "#6" $B_{3.3.ref}$ = "EW Bot Perim. Reinf." $B_{3.4}$ = "#4" $B_{3.4.ref}$ = "SW Bot Perim. Reinf."

$N_{B.3.3} = 1$ $N_{B.3.3.ref}$ = "# EW Bot Perim. Bars" $N_{B.3.4} = 2$ $N_{B.3.4.ref}$ = "# SW Bot Perim. Bars"

$D_{3c.3} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.4} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.3} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.4} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.5}$ = "#5" $B_{3.5.ref}$ = "Wall Opening Reinf." $D_{3c.5} = 0.625 \cdot in$ D_{ref} = "Bar Diam."

$N_{B.3.5} = 2$ $N_{B.3.5.ref}$ = "# Wall Opening Bars" $A_{3c.5} = 0.62 \cdot in^2$ A_{ref} = "Area of Reinf."

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 100 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 26.2 \;\; -26.2 \;\; 213.1 \;\; 213.1 \;\; 26.2 \;\; -26.2) \cdot plf$

$P_{u.3c.h1d1}{}^T = (4.75 \;\; 5.27 \;\; 7.91 \;\; 7.91 \;\; 9.2 \;\; 5.27 \;\; 6.56 \;\; 4.43 \;\; 4.02 \;\; 2.71 \;\; 5.28 \;\; 2.91 \;\; 2.5) \cdot kip$

$M_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.16 \;\; -0.16 \;\; 1.31 \;\; 1.31 \;\; 0.16 \;\; -0.16) \cdot kip \cdot ft$

$V_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.09 \;\; -0.09 \;\; 0.75 \;\; 0.75 \;\; 0.09 \;\; -0.09) \cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d1.mx} = 9.2 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.31 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.46 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.5 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.16 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 100 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$W_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 19.7 \;\; -19.7 \;\; 159.8 \;\; 159.8 \;\; 19.7 \;\; -19.7)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.56 \;\; 3.95 \;\; 5.93 \;\; 5.93 \;\; 6.9 \;\; 3.95 \;\; 4.92 \;\; 3.32 \;\; 3.02 \;\; 2.03 \;\; 3.96 \;\; 2.18 \;\; 1.88)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.12 \;\; -0.12 \;\; 0.98 \;\; 0.98 \;\; 0.12 \;\; -0.12)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.07 \;\; -0.07 \;\; 0.56 \;\; 0.56 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 6.9\cdot kip$ $M_{u.3c.h1d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.09\cdot kip$

$P_{u.3c.h1d2.mn} = 1.88\cdot kip$ $M_{u.3c.h1d2.mn} = -0.12\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 16.4 \;\; -16.4 \;\; 133.2 \;\; 133.2 \;\; 16.4 \;\; -16.4)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.97 \;\; 3.29 \;\; 4.94 \;\; 4.94 \;\; 5.75 \;\; 3.29 \;\; 4.1 \;\; 2.77 \;\; 2.51 \;\; 1.69 \;\; 3.3 \;\; 1.82 \;\; 1.57)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.1 \;\; -0.1 \;\; 0.82 \;\; 0.82 \;\; 0.1 \;\; -0.1)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.06 \;\; -0.06 \;\; 0.47 \;\; 0.47 \;\; 0.06 \;\; -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 5.75\cdot kip$ $M_{u.3c.h1d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 0.91\cdot kip$

$P_{u.3c.h1d3.mn} = 1.57\cdot kip$ $M_{u.3c.h1d3.mn} = -0.1\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 26.2 \;\; -26.2 \;\; 213.1 \;\; 213.1 \;\; 26.2 \;\; -26.2)\cdot plf$

$P_{u.3c.h2d1}^{T} = (5 \;\; 5.49 \;\; 8.13 \;\; 8.13 \;\; 9.42 \;\; 5.49 \;\; 6.78 \;\; 4.65 \;\; 4.24 \;\; 2.85 \;\; 5.43 \;\; 3.05 \;\; 2.65)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.16 \;\; -0.16 \;\; 1.31 \;\; 1.31 \;\; 0.16 \;\; -0.16)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.09 \;\; -0.09 \;\; 0.75 \;\; 0.75 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 9.42\cdot kip$ $M_{u.3c.h2d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.46\cdot kip$

$P_{u.3c.h2d1.mn} = 2.65\cdot kip$ $M_{u.3c.h2d1.mn} = -0.16\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 19.7 \;\; -19.7 \;\; 159.8 \;\; 159.8 \;\; 19.7 \;\; -19.7)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.75 \;\; 4.12 \;\; 6.1 \;\; 6.1 \;\; 7.06 \;\; 4.12 \;\; 5.08 \;\; 3.48 \;\; 3.18 \;\; 2.14 \;\; 4.07 \;\; 2.29 \;\; 1.98)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.12 \;\; -0.12 \;\; 0.98 \;\; 0.98 \;\; 0.12 \;\; -0.12)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.07 \;\; -0.07 \;\; 0.56 \;\; 0.56 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 7.06\cdot kip$ $M_{u.3c.h2d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.09\cdot kip$

$P_{u.3c.h2d2.mn} = 1.98\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.12\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 16.4 \;\; -16.4 \;\; 133.2 \;\; 133.2 \;\; 16.4 \;\; -16.4)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (3.13 \;\; 3.43 \;\; 5.08 \;\; 5.08 \;\; 5.89 \;\; 3.43 \;\; 4.24 \;\; 2.9 \;\; 2.65 \;\; 1.78 \;\; 3.39 \;\; 1.91 \;\; 1.65)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.1 \;\; -0.1 \;\; 0.82 \;\; 0.82 \;\; 0.1 \;\; -0.1)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.06 \;\; -0.06 \;\; 0.47 \;\; 0.47 \;\; 0.06 \;\; -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 5.89\cdot kip$ $M_{u.3c.h2d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 0.91\cdot kip$

$P_{u.3c.h2d3.mn} = 1.65\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.1\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 26.2 \;\; -26.2 \;\; 213.1 \;\; 213.1 \;\; 26.2 \;\; -26.2)\cdot plf$

$P_{u.3c.h3d1}^{T} = (5.17 \;\; 5.63 \;\; 8.27 \;\; 8.27 \;\; 9.56 \;\; 5.63 \;\; 6.92 \;\; 4.79 \;\; 4.38 \;\; 2.94 \;\; 5.52 \;\; 3.15 \;\; 2.74)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.16 \;\; -0.16 \;\; 1.31 \;\; 1.31 \;\; 0.16 \;\; -0.16)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.09 \;\; -0.09 \;\; 0.75 \;\; 0.75 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 9.56\cdot kip$ $M_{u.3c.h3d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.46\cdot kip$

$P_{u.3c.h3d1.mn} = 2.74\cdot kip$ $M_{u.3c.h3d1.mn} = -0.16\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 19.7 \;\; -19.7 \;\; 159.8 \;\; 159.8 \;\; 19.7 \;\; -19.7)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.88 \;\; 4.23 \;\; 6.21 \;\; 6.21 \;\; 7.17 \;\; 4.23 \;\; 5.19 \;\; 3.59 \;\; 3.29 \;\; 2.21 \;\; 4.14 \;\; 2.36 \;\; 2.06)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.12 \;\; -0.12 \;\; 0.98 \;\; 0.98 \;\; 0.12 \;\; -0.12)\cdot kip\cdot ft$

$V_{u.3c.h3d2}^{T} = (0 \quad 0 \quad 0 \quad 0.28 \quad 0.28 \quad 0.56 \quad 0.28 \quad 0.07 \quad -0.07 \quad 0.56 \quad 0.56 \quad 0.07 \quad -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 7.17\cdot kip$ $M_{u.3c.h3d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.09\cdot kip$

$P_{u.3c.h3d2.mn} = 2.06\cdot kip$ $M_{u.3c.h3d2.mn} = -0.12\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67 ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3 ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \quad 0 \quad 0 \quad 66.6 \quad 66.6 \quad 133.2 \quad 66.6 \quad 16.4 \quad -16.4 \quad 133.2 \quad 133.2 \quad 16.4 \quad -16.4)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \quad 0 \quad 0 \quad 0.41 \quad 0.41 \quad 0.82 \quad 0.41 \quad 0.1 \quad -0.1 \quad 0.82 \quad 0.82 \quad 0.1 \quad -0.1)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \quad 0 \quad 0 \quad 0.23 \quad 0.23 \quad 0.47 \quad 0.23 \quad 0.06 \quad -0.06 \quad 0.47 \quad 0.47 \quad 0.06 \quad -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 5.98\cdot kip$ $M_{u.3c.h3d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 0.91\cdot kip$

$P_{u.3c.h3d3.mn} = 1.71\cdot kip$ $M_{u.3c.h3d3.mn} = -0.1\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - (0.75in + 0.5D_{3c.5})\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.456\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.092\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 0.91\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{(k_{wall}\cdot H_{wall})^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

$\delta_{3c.h1d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.469$ $\Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d1} = 0.681$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

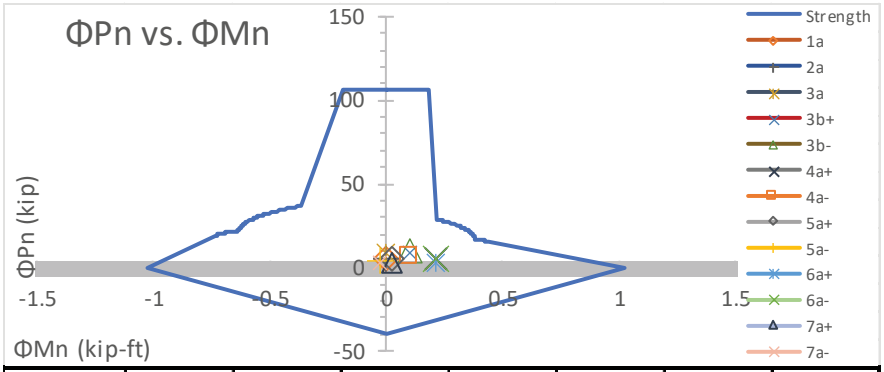
$\delta_{3c.h2d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d1} = 1.677$ $\Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1}$ $\Omega_{\delta.3c.h2d1} = 0.596$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h3d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d1} = 3.326$ $\Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1}$ $\Omega_{\delta.3c.h3d1} = 0.301$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3c.d1} := \min(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1})$ $\Omega_{\delta.3c.d1} = 0.301$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.9%	5.3%	7.8%	15.17%	16.3%	26.5%	14.1%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.5%	4.1%	23.4%	26.3%	3.0%	2.7%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3} = \text{"OK"}$

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h1d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.469$ $\Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d2} = 0.681$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h2d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d2} = 1.434$ $\Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1}$ $\Omega_{\delta.3c.h2d2} = 0.697$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h3d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d2} = 2.103$ $\Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1}$ $\Omega_{\delta.3c.h3d2} = 0.476$

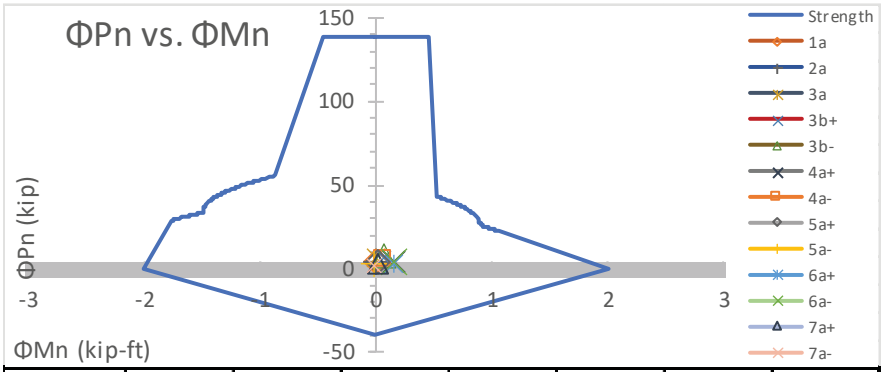
Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\Omega_{\delta.3c.d2} := \min(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2})$ $\Omega_{\delta.3c.d2} = 0.476$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.8%	3.1%	4.5%	4.59%	5.2%	8.8%	4.5%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.6%	2.4%	8.4%	8.7%	1.7%	1.5%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.249 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.8$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

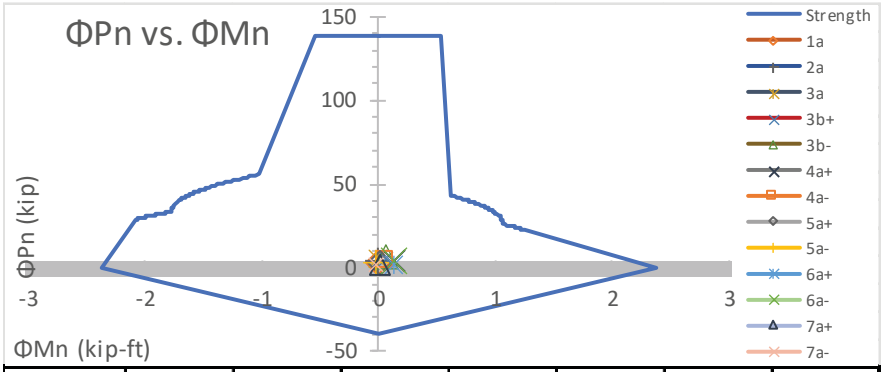
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.337 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.748$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.776 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.563$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.563$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.3%	2.5%	3.7%	3.74%	4.3%	6.1%	3.1%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.2%	2.0%	5.8%	6.1%	1.4%	1.2%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

d. Wall - Header Check (Above Opening) $w_{llr} = 100 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in$ Min Header Height for Opening Widths of $w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$Wu_{3d.0}^{T} = (827.9 \quad 1009.6 \quad 1669.6 \quad 1669.6 \quad 1991.7 \quad 1009.6 \quad 1331.7 \quad 798.7 \quad 696.8 \quad 476.4 \quad 1120.6 \quad 527.4 \quad 425.5) \cdot plf$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1991.7 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in$ $A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for for:
$w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min\left(Vc_{3d.0.a}, Vc_{3d.0.b}\right)$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot \left(w_{open_0}\right)^2}{8}$ $Mu_{3d.0} = 8.96 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max\left(\left|wu_{3d.0mx}\right|, \left|wu_{3d.0mn}\right|\right) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 5.98 \cdot kip$

$chk_{3d.1} := \text{if}\left(\phi Mn_{3d.0} \geq Mu_{3d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot \left[f_y \cdot \left(0.5 \cdot h_{3d_0}\right)\right]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := \text{if}\left(chk_{3d.2} = \text{"OK"}, \text{"OK"}, \text{if}\left(chk_{3d.2} = \text{"Stirrups Req'd"}, \text{if}\left(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, \text{"NG"}, \text{"OK"}\right), \text{"OK"}\right)\right)$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (727.4 \quad 923.5 \quad 1583.5 \quad 1583.5 \quad 1905.6 \quad 923.5 \quad 1245.6 \quad 712.6 \quad 610.7 \quad 420.4 \quad 1064.6 \quad 471.3 \quad 369.4) \cdot plf$

$wu_{3d.1mx} := \max\left(Wu_{3d.1}\right)$ $wu_{3d.1mx} = 1905.6 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min\left(Wu_{3d.1}, 0plf\right)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}\left(a_{3d.1.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots + A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := \text{root}\left(f_{3d.1}\left(a_{3d.1.g}\right), a_{3d.1.g}\right)$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F2_{3d.1} := A_{3d.1_1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.1} = 11.7\cdot kip$$

$$F3_{3d.1} := A_{3d.1_2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.1} = 7.2\cdot kip$$

$$F4_{3d.1} := A_{3d.1_3}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.1} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772\cdot kip$$

$$Mn_{3d.1} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.1}\cdot\frac{a_{3d.1}}{2} + F1_{3d.1}\cdot d_{3d.1_0} + F2_{3d.1}\cdot d_{3d.1_1} \ldots + F3_{3d.1}\cdot d_{3d.1_2} + F4_{3d.1}\cdot d_{3d.1_3} \qquad Mn_{3d.1} = 38.5\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}}\cdot d_{3d.1_3} \qquad \varepsilon_{3d.1} = 0.014 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.1} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.1} := \phi_{3d.1}\cdot Mn_{3d.1} \qquad \phi Mn_{3d.1} = 34.64\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375\cdot in$$

$$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right)\cdot t_w} \qquad \rho_{3d.1} = 0.0062$$

$$\rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{3d.1} := \sum\begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.1_0}\right)}\right] \qquad \rho'_{3d.1} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.1} = 0.876$$

$$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$$

$$Vc_{3d.1.a} := 8\cdot\lambda_{s.3d.1}\cdot\lambda\cdot\rho_{3d.1}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.a} = 5.48\cdot kip$$

$$Vc_{3d.1.b} = 21.31\cdot kip$$

$$\phi Vc_{3d.1} := \phi_V\cdot\min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right) \qquad \phi Vc_{3d.1} = 4.1\cdot kip \qquad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$$

$$Mu_{3d.1} := \frac{wu_{3d.1mx}\cdot\left(w_{open_1}\right)^2}{8} \qquad Mu_{3d.1} = 3.81\cdot kip\cdot ft$$

$$Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right)\cdot w_{open_1}}{2} \qquad Vu_{3d.1} = 3.81\cdot kip$$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h_{3d_1}\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (706.3 \quad 905.4 \quad 1565.4 \quad 1565.4 \quad 1887.5 \quad 905.4 \quad 1227.5 \quad 694.4 \quad 592.5 \quad 408.6 \quad 1052.8 \quad 459.5 \quad 357.6) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1887.5 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_2} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_2} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 3.81 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 2.83 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2} \\ \text{"Min. Stirrups Req'd"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2} \\ \text{"Stirrups Req'd"} & \text{otherwise} \end{cases}$ $\quad chk_{3d.10} = \text{"Min. Stirrups Req'd"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 2.74 \cdot psf$		$M_{E.4a} = 0.001 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.002 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.234 \\ 0.234 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.08 \\ 0.08 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.193 \\ 0.193 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.063 \\ 0.063 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.151 \\ 0.151 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PSF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
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b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 8.45 \cdot plf$		$M_{E.4b} = 0.092 \cdot kip \cdot ft$	$V_{E.4b} = 0.039 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.49 \\ 6.49 \\ 0.58 \\ 0.58 \\ 0.67 \\ 0.67 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.78 \\ 2.78 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.4 \\ 5.4 \\ 0.58 \\ 0.58 \\ 0.67 \\ 0.67 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.31 \\ 2.31 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.31 \\ 4.31 \\ 0.58 \\ 0.58 \\ 0.67 \\ 0.67 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.85 \\ 1.85 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

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Plan No.: MFT-4193-Standard Concrete
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Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

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Const. Type: VB
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

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$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (12 \quad 13 \quad 14 \quad 15)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (29.29 \quad 31.29 \quad 33.29 \quad 35.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (6.86 \quad 6.96 \quad 7.04 \quad 7.12) \cdot psf$ Floor Transverse Ribs

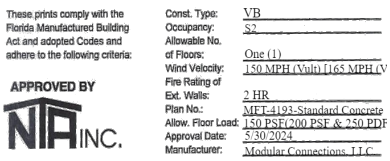
$wgt_{f.l.rib}^T = (3.63 \quad 3.63 \quad 3.64 \quad 3.64) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (3.27 \quad 3.06 \quad 2.88 \quad 2.72) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (4.13 \quad 4.15 \quad 4.16 \quad 4.17) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (37.6 \quad 37.5 \quad 37.4 \quad 37.3) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (41.6 \quad 41.5 \quad 41.4 \quad 41.3) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"



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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

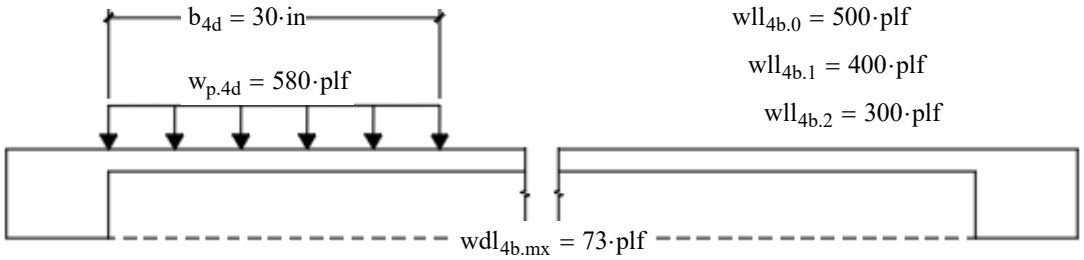
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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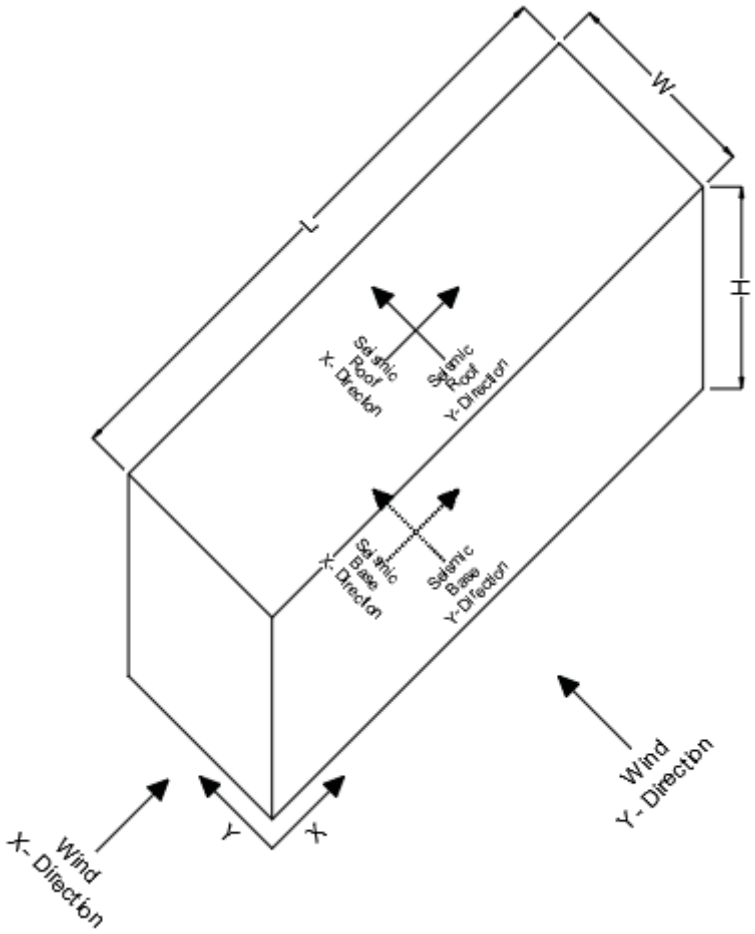
5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx} \cdot (L_r) \cdot (W_r)$

$Wt_{rf.mn} := DL_{r.mn} \cdot (L_r) \cdot (W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25 \cdot w_{llr}, 0psf)$

$w_{llr.eq} = 25 \cdot psf$

$Wt_{25ll} := w_{llr.eq} \cdot [(L_r) \cdot (W_r)]$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{rf.mx}^{T} = (26.7 \quad 28.5 \quad 30.2 \quad 32) \cdot kip$

Max Roof Weight, $w_{llr} = 100 \cdot psf$

$Wt_{25ll}^{T} = (9.1 \quad 9.7 \quad 10.3 \quad 10.9) \cdot kip$

25% Roof LLr 0, $w_{llr} = 100 \cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (36 \quad 38 \quad 41 \quad 43) \cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (24.2 \quad 25.8 \quad 27.3 \quad 28.9) \cdot kip$

Min Roof Weight, $w_{llr} = 100 \cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75 \cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3} \cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128) \cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (115.5 \quad 127.5 \quad 135.5) \cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 111.75 \\ 123.75 \\ 131.75 \end{pmatrix} \cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r} \cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r} \cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r} \cdot el_{avg.rf_2}$

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Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h1.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h2.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h0.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h1.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h2.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{EQ.w.h0}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h1}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h2}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

Max Wall Weight, $H_{wall_0} = 9\,ft$

Max Wall Weight, $H_{wall_1} = 10\,ft$

Max Wall Weight, $H_{wall_2} = 10.7\,ft$

Min Wall Weight, $H_{wall_0} = 9\,ft$

Min Wall Weight, $H_{wall_1} = 10\,ft$

Min Wall Weight, $H_{wall_2} = 10.7\,ft$

Seismic Wall Weight, $H_{wall_0} = 9\,ft$

Seismic Wall Weight, $H_{wall_1} = 10\,ft$

Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$el_{top.w} := h_{wl\_fl} + H_{wall}$ $\quad el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$ Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean(h_{wl\_fl}, el_{top.w_0}) \\ mean(h_{wl\_fl}, el_{top.w_1}) \\ mean(h_{wl\_fl}, el_{top.w_2}) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$ $\quad W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$ $\quad W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf)$ $\quad w_{llf.0.eq} = 62.5 \cdot psf$ $\quad Wt_{25llf0} := w_{llf.0.eq} \cdot [(L_f) \cdot (W_f)]$

$w_{llf.1.eq} := if(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf)$ $\quad w_{llf.1.eq} = 50 \cdot psf$ $\quad Wt_{25llf1} := w_{llf.0.eq} \cdot [(L_f) \cdot (W_f)]$

$w_{llf.2.eq} := if(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf)$ $\quad w_{llf.2.eq} = 37.5 \cdot psf$ $\quad Wt_{25llf2} := w_{llf.0.eq} \cdot [(L_f) \cdot (W_f)]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$ $\quad Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (13.3 \quad 14.2 \quad 15.1 \quad 16) \cdot kip$

Max Floor Weight

$Wt_{f.mn}^T = (12.1 \quad 12.8 \quad 13.6 \quad 14.4) \cdot kip$

Min Floor Weight

$Wt_{25llf0}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$ $\quad Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$ $\quad Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

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$Wt_{EQ.f.f0}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 45.99\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (13 \quad 13.9 \quad 14.8 \quad 15.7)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (14.3 \quad 15.3 \quad 16.3 \quad 17.3)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (15.2 \quad 16.3 \quad 17.3 \quad 18.4)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 5.17\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 5.71\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 6.07\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (6.5 \quad 6.9 \quad 7.4 \quad 7.8)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (7.2 \quad 7.7 \quad 8.1 \quad 8.6)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (7.6 \quad 8.1 \quad 8.7 \quad 9.2)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 2.59\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 2.86\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 3.03\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.145$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{EQ.bh0f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh1f0} := C_s \cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.bh2f0}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh2f0} := C_s \cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.bh0f1}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh0f1} := C_s \cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

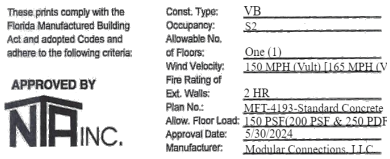
$Wt_{EQ.bh1f1}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh1f1} := C_s \cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$



$Wt_{EQ.bh2f1}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh2f1} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.bh0f2}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh0f2} := C_s \cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.bh1f2}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh1f2} := C_s \cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.bh2f2}^{T} = (103 \quad 109 \quad 115 \quad 121)\cdot kip$

$V_{EQ.bh2f2} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.rh0}^{T} = (53 \quad 56 \quad 59 \quad 62)\cdot kip$

$V_{EQ.rh0} := C_s \cdot Wt_{EQ.rh0}$

Roof Diap. Wt/Shear

$w_{llr} = 100\cdot psf \quad H_{wall_0} = 9\,ft$

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$V_{EQ.rh0}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

$Wt_{EQ.rh1}^{T} = (53 \quad 56 \quad 59 \quad 62)\cdot kip$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

$Wt_{EQ.rh2}^{T} = (53 \quad 56 \quad 59 \quad 62)\cdot kip$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

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Manufacturer:	Modular Connections, LLC

Roof Diap. Wt/Shear
$w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

Roof Diap. Wt/Shear
$w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.7\,ft$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r} \qquad w_{EQh0.y}^{T} = (250 \quad 249 \quad 248 \quad 247)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r} \qquad w_{EQh1.y}^{T} = (250 \quad 249 \quad 248 \quad 247)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r} \qquad w_{EQh2.y}^{T} = (250 \quad 249 \quad 248 \quad 247)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r} \qquad w_{EQh0.x}^{T} = (633 \quad 671 \quad 710 \quad 748)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r} \qquad w_{EQh1.x}^{T} = (633 \quad 671 \quad 710 \quad 748)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r} \qquad w_{EQh2.x}^{T} = (633 \quad 671 \quad 710 \quad 748)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r} \qquad w_{W.h0.y}^{T} = (214 \quad 215 \quad 215 \quad 215)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r} \qquad w_{W.h1.y}^{T} = (236 \quad 237 \quad 237 \quad 238)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r} \qquad w_{W.h2.y}^{T} = (251 \quad 252 \quad 252 \quad 253)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67\,ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$

$w_{W.h0.x} = 215.6\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$

$w_{W.h1.x} = 238\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$

$w_{W.h2.x} = 252.9\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_2} = 10.7\,ft$

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$M_{u.bh0.y}^{T} = (28.8 \quad 32.6 \quad 36.6 \quad 40.8)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (28.8 \quad 32.6 \quad 36.6 \quad 40.8)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (28.8 \quad 32.6 \quad 36.6 \quad 40.8)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.y}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.bh0.x}^{T} = (11.4 \quad 12.1 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (11.4 \quad 12.1 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (11.4 \quad 12.1 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.x}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.b.h0.y}^{T} = (24.6 \quad 28 \quad 31.7 \quad 35.5)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (27.2 \quad 31 \quad 35 \quad 39.2)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (28.9 \quad 32.9 \quad 37.2 \quad 41.7)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{u.b.h0.y}^{T} = (3.2 \;\; 3.5 \;\; 3.7 \;\; 3.9)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (3.6 \;\; 3.8 \;\; 4.1 \;\; 4.3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (3.8 \;\; 4.1 \;\; 4.3 \;\; 4.6)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 3.9\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 4.3\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 4.6\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

$v_{u.b.h0.x} = 1.3\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.4\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = \text{"\#6"}$ $B_{7.1.ref} = \text{"Roof Perim. Reinf."}$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = \text{"\# Roof Perim. Bars"}$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = \text{"Roof Perim. Reiinf. Bar Diam."}$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = \text{"Roof Perim. Reinf. Area"}$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := \text{"Roof Diaph.Reinf."}$ $\rho_{b.1} := \dfrac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00074$

$d_{b.1} := W_r - 0.75in - \dfrac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \dfrac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 1.67\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \dfrac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\dfrac{d_{b.1}}{2} - \dfrac{a_{b.1}}{2}\right)$ $M_{b.1} = 383.1\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \dfrac{a_{b.1}}{0.85}$ $c_{b.1} = 2\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\dfrac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.21474$ Strain in Ten. Rebar

$\phi_{b.1} := \text{if}\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 344.8\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\dfrac{M_{u.bh0.y}}{\phi M_{b.1_0}} \;\; \dfrac{M_{u.bh1.y}}{\phi M_{b.1_1}} \;\; \dfrac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \;\; 10 \;\; 10.7)\,ft$ & $w_{llr} = 100\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 8.4 & 9.4 & 10.6 & 11.8 \\ 8.4 & 9.4 & 10.6 & 11.8 \\ 8.4 & 9.4 & 10.6 & 11.8 \end{pmatrix}\cdot\%$

$chk_{b.1a} := \text{if}\left(\max\left(Unity_{b.1a}\right) < 1, \text{"OK"}, \text{"NG"}\right)$

$chk_{b.1a} = \text{"OK"}$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.1b}^{T} = \begin{pmatrix} 7.1 & 8.1 & 9.2 & 10.3 \\ 7.9 & 9 & 10.1 & 11.4 \\ 8.4 & 9.5 & 10.8 & 12.1 \end{pmatrix} \cdot \%$$

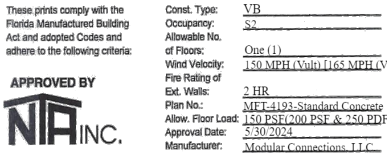
$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.68 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 1.673 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$



$$d_{b.2}^{T} = (30.26 \quad 32.26 \quad 34.26 \quad 36.26)\,ft$$

$$\rho_{b.2}^{T} = (0.00031 \quad 0.00029 \quad 0.00028 \quad 0.00026)$$

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.1}}{2} \right)$$

$$M_{b.2a}^{T} = (1014 \quad 1081 \quad 1148 \quad 1215) \cdot kip \cdot ft$$

Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 2.1 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$$\varepsilon_{t.b.2a}^{T} = (0.5179 \quad 0.5523 \quad 0.5867 \quad 0.6211)$$

Strain in Rebar

$\phi_{b.2} := if\left[min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50 \right] \right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^{T} = (912 \quad 973 \quad 1033 \quad 1094) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.2a}^{T} = \begin{pmatrix} 1.2 & 1.2 & 1.2 & 1.2 \\ 1.2 & 1.2 & 1.2 & 1.2 \\ 1.2 & 1.2 & 1.2 & 1.2 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.2b}^{T} = \begin{pmatrix} 0.4 & 0.4 & 0.4 & 0.4 \\ 0.5 & 0.4 & 0.4 & 0.4 \\ 0.5 & 0.5 & 0.4 & 0.4 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 864 \cdot in^2$ Roof Shear Area for $t_{r.e} = 6 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00074$

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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e} \cdot L_{r_0} \\ t_{r.e} \cdot L_{r_1} \\ t_{r.e} \cdot L_{r_2} \\ t_{r.e} \cdot L_{r_3} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 2184 \\ 2328 \\ 2472 \\ 2616 \end{pmatrix} \cdot in^2$$

Roof Shear Area for $t_{r.e} = 6 \cdot in$ & $L_r = \begin{pmatrix} 4368 \\ 4656 \\ 4944 \\ 5232 \end{pmatrix} \cdot \frac{in^2}{ft}$

$\rho_{b.2}^{T} = (0.00031 \quad 0.00029 \quad 0.00028 \quad 0.00026)$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv} \cdot \left(2 \cdot \sqrt{f'_c} + \rho \cdot f_y\right)$ $V_{b.3a.1.mx} = 154.3 \cdot kip$

Max Allowable for $t_{r.e} = 6 \cdot in$ & $W_r = 12 \cdot ft$

$V_{b.3a.2.mx}^{T} = (333.8 \quad 353.1 \quad 372.5 \quad 391.8) \cdot kip$

Max Allowable Shear for $t_{r.e} = 6 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$V_{b.3b.1} = 49.3 \cdot kip$ Nominal Shear Strength for $t_{r.e} = 6 \cdot in$ & $W_r = 12 \cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

Roof Diaph. Shear Strength for $t_{r.e} = 6 \cdot in$ & $W_r = 12 \cdot ft$ $\phi V_{c.b.3a.1} = 36.9 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$V_{b.3b.2}^{T} = (124.5 \quad 132.7 \quad 141 \quad 149.2) \cdot kip$

Nominal Shear Strength for $t_{r.e} = 6 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$\phi V_{c.b.3a.2}^{T} = (93.4 \quad 99.6 \quad 105.7 \quad 111.9) \cdot kip$

Roof Diaph. Shear Strength for $t_{r.e} = 6 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 10.3 & 10.9 & 11.5 & 12.1 \\ 10.3 & 10.9 & 11.5 & 12.1 \\ 10.3 & 10.9 & 11.5 & 12.1 \end{pmatrix} \cdot \%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 8.8 & 9.4 & 10 & 10.6 \\ 9.7 & 10.4 & 11 & 11.7 \\ 10.3 & 11 & 11.7 & 12.4 \end{pmatrix} \cdot \%$$

$chk_{b.3d} = "OK"$

Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 100 \cdot psf$

$Unity_{b.3g}^{T} = (4.1 \quad 4 \quad 4 \quad 4) \cdot \%$

$chk_{b.3j} = "OK"$

Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 1.4 & 1.3 & 1.2 & 1.2 \\ 1.5 & 1.4 & 1.4 & 1.3 \\ 1.6 & 1.5 & 1.4 & 1.4 \end{pmatrix} \cdot \%$$

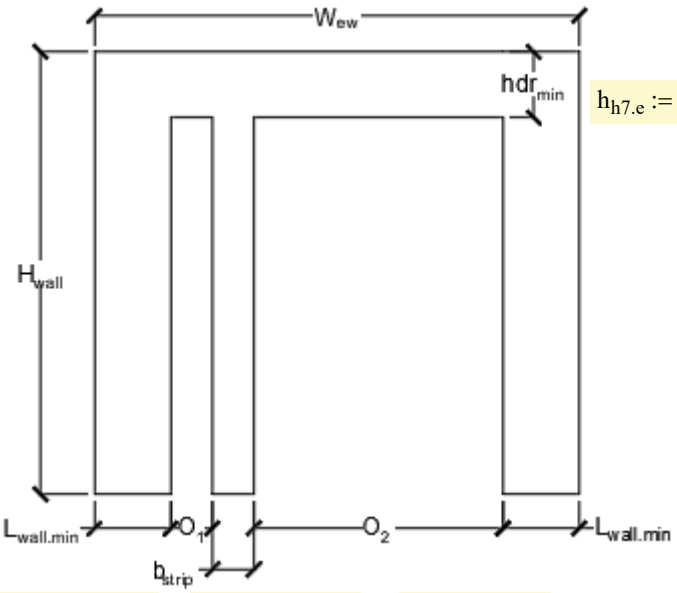
$chk_{b.3j} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

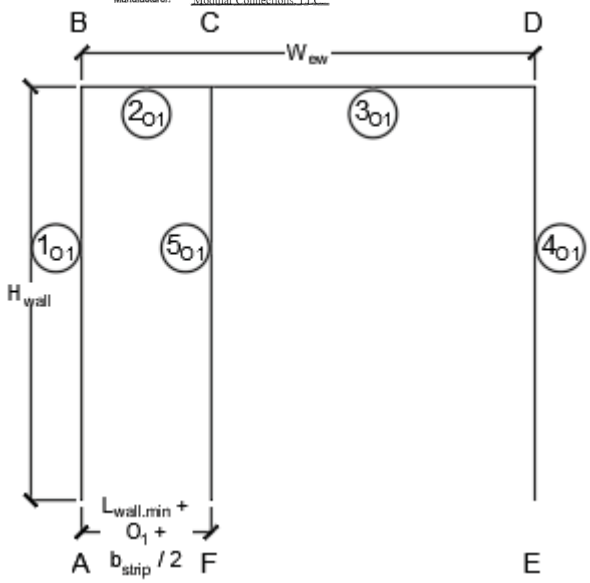
APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

END WALL DESIGN



$h_{h7.e} := 19in$



$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} =$ "Modulus of Elas. Conc."

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right) \qquad Veq_{ewho}^{T} = (3.8 \;\; 4 \;\; 4.3 \;\; 4.5) \cdot kip \qquad \text{Height } H_{wall_0} = 9\,ft$$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right) \qquad Veq_{ewh1}^{T} = (3.8 \;\; 4 \;\; 4.3 \;\; 4.5) \cdot kip \qquad \text{Height } H_{wall_1} = 10\,ft$$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right) \qquad Veq_{ewh2}^{T} = (3.8 \;\; 4 \;\; 4.3 \;\; 4.5) \cdot kip \qquad \text{Height } H_{wall_2} = 10.7\,ft$$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix} \qquad K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \dfrac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \dfrac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint C

$$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix} \qquad K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \dfrac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \dfrac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

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Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}{}^T = (6.4 \quad 6.8 \quad 7.2 \quad 7.6) \cdot kip \cdot ft$

$V_{L.ewh0}{}^T = (1.2 \quad 1.3 \quad 1.4 \quad 1.5) \cdot kip$

$M_{hdrh0}{}^T = (4.7 \quad 5 \quad 5.3 \quad 5.6) \cdot kip \cdot ft$

$V_{hdrh0}{}^T = (1.6 \quad 1.7 \quad 1.8 \quad 1.9) \cdot kip$

$M_{b.striph0}{}^T = (7.1 \quad 7.5 \quad 7.9 \quad 8.3) \cdot kip \cdot ft$

$V_{b.striph0}{}^T = (1.5 \quad 1.5 \quad 1.6 \quad 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10 ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

$DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}{}^T = (7.4 \quad 7.8 \quad 8.3 \quad 8.7) \cdot kip \cdot ft$

$V_{L.ewh1}{}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$M_{hdrh1}^{T} = (5.9 \quad 6.2 \quad 6.6 \quad 6.9)\cdot kip\cdot ft$

$V_{hdrh1}^{T} = (2.6 \quad 2.8 \quad 2.9 \quad 3.1)\cdot kip$

$M_{b.striph1}^{T} = (7.9 \quad 8.4 \quad 8.9 \quad 9.3)\cdot kip\cdot ft$

$V_{b.striph1}^{T} = (1.5 \quad 1.6 \quad 1.7 \quad 1.2)\cdot kip$

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4\cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4\cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix}\cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119\cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh2}^{T} = (7.9 \quad 8.5 \quad 9 \quad 9.5)\cdot kip\cdot ft$

$V_{L.ewh2}^{T} = (1.3 \quad 1.4 \quad 1.5 \quad 1.6)\cdot kip$

$M_{hdrh2}^{T} = (6.4 \quad 6.8 \quad 7.3 \quad 7.7)\cdot kip\cdot ft$

$V_{hdrh2}^{T} = (2.9 \quad 3 \quad 3.2 \quad 3.4)\cdot kip$

$M_{b.striph2}^{T} = (8.5 \quad 9 \quad 9.6 \quad 10.2)\cdot kip\cdot ft$

$V_{b.striph2}^{T} = (1.5 \quad 1.6 \quad 1.7 \quad 1.2)\cdot kip$

Reinforcement

$B_{3.1} = "\#6"$ $B_{3.1.ref} = "Wall\ Vert.\ Perim.\ Reinf."$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = "\#\ Wall\ Vert.\ Perim.\ Bars"$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.2} = "\#4"$ $B_{3.2.ref} = "Wall\ Top.\ Perim.\ Reinf."$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = "\#\ Wall\ Top\ Perim.\ Bars"$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.3} = "\#6"$ $B_{3.3.ref} = "EW\ Bot\ Perim.\ Reinf."$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = "\#\ EW\ Bot\ Perim.\ Bars"$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.4} = "\#4"$ $B_{3.4.ref} = "SW\ Bot\ Perim.\ Reinf."$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = "\#\ SW\ Bot\ Perim.\ Bars"$

$D_{3.4} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17 \cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5 \cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5 \cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{ewc} = 1.7 \cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{h.e} = 1.6 \cdot in$

$a_{strip} := \frac{A_{3.5} \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{strip} = 2.4 \cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot L_{ew.mn}$ $A_{mn.e} = 0.079 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.h.e} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.strip} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w \cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w \cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w \cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

$chk_{c.1} := \text{if}(A_{mn.e} \le \min(A_{3.1}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.1} = \text{"OK"}$

$chk_{c.2} := \text{if}(A_{mn.e} \le \min(A_{3.2}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.2} = \text{"OK"}$

$chk_{c.3} := \text{if}(A_{mn.strip} \le A_{3.5}, \text{"OK"}, \text{"NG"})$ $chk_{c.3} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{ewc} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{h.e} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{strip} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi Mn_{ewc} := \phi_{ewc} \cdot \min(A_{3.1}, A_{3.5}) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min(A_{3.2}, A_{3.5}) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref}$ = "Flexural Strength, +ve, Sect (22.3)"

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{ewc.1} := 8 \cdot \lambda \cdot \lambda_{s.ewc} \cdot \rho_{ewc}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975 \cdot kip$ $Vc_{ewc.2} = 19.387 \cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V \cdot \min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2 \cdot \left(t_w \cdot d_{ewc}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{ewc} = 3.7 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8 \cdot \lambda \cdot \lambda_{s.h.e} \cdot \rho_{h.e}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663 \cdot kip$ $Vc_{h.e.2} = 17.676 \cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V \cdot \min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.e} = 3.5 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8 \cdot \lambda \cdot \lambda_{s.strip}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347 \cdot kip$ $Vc_{strip.2} = 26.799 \cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V \cdot \min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.33 & 0.35 & 0.36 & 0.2 & 0.23 & 0.25 \\ 0.35 & 0.38 & 0.38 & 0.21 & 0.24 & 0.26 \\ 0.37 & 0.4 & 0.41 & 0.22 & 0.26 & 0.28 \\ 0.39 & 0.42 & 0.43 & 0.24 & 0.27 & 0.3 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

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$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{strip} = \begin{pmatrix} 0.17 & 0.18 & 0.18 & 0.11 & 0.13 & 0.14 \\ 0.19 & 0.19 & 0.19 & 0.12 & 0.13 & 0.15 \\ 0.2 & 0.2 & 0.2 & 0.13 & 0.14 & 0.15 \\ 0.16 & 0.14 & 0.14 & 0.13 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{hdr} = \begin{pmatrix} 0.47 & 0.75 & 0.82 & 0.18 & 0.22 & 0.24 \\ 0.5 & 0.79 & 0.87 & 0.19 & 0.24 & 0.26 \\ 0.52 & 0.84 & 0.93 & 0.2 & 0.25 & 0.27 \\ 0.55 & 0.88 & 0.98 & 0.21 & 0.26 & 0.29 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (3.8\ \ 4\ \ 4.3\ \ 4.5)\cdot kip$ $Vuh1^T = (3.8\ \ 4\ \ 4.3\ \ 4.5)\cdot kip$ $Vuh2^T = (3.8\ \ 4.1\ \ 4.3\ \ 4.6)\cdot kip$

Wall Plates: $No_{plates} := 1$ $V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$ $V_{u.Floor.PLh0}^T = (5.9\ \ 6.2\ \ 6.6\ \ 6.9)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$ $V_{u.Floor.PLh1}^T = (6.5\ \ 6.9\ \ 7.3\ \ 7.7)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$ $V_{u.Floor.PLh2}^T = (7\ \ 7.4\ \ 7.9\ \ 8.4)\cdot kip$

$A_{cv.b.3b.1}^T = (2184\ \ 2328\ \ 2472\ \ 2616)\cdot in^2$

$\rho_{b.2}^T = (0.00031\ \ 0.00029\ \ 0.00028\ \ 0.00026)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \Longrightarrow 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force

$v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.1.2h0}^T = (3.8\ \ 4\ \ 4.3\ \ 4.5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.2.2h1}^T = (4.2\ \ 4.5\ \ 4.7\ \ 5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.3.2h2} := \dfrac{V_{u.Wall.PLh2}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.3.2h2}^T = (4.5\ \ 4.8\ \ 5.1\ \ 5.4)\cdot\dfrac{kip}{ft}$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.2.2h1}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.3.2h2}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T\right)^T \qquad Hwall := \left(H_{wall_0}\ \ H_{wall_1}\ \ H_{wall_2}\right)$$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Hwall = (9.00 \quad 10.00 \quad 10.67)\,ft$

$$Unity_{5d1.2.3} = \begin{pmatrix} 59.04 & 65.6 & 70.16 \\ 62.61 & 69.56 & 74.95 \\ 66.18 & 73.53 & 79.74 \\ 69.75 & 77.5 & 84.54 \end{pmatrix} \cdot \%\ L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

$chk_{5d.1} := if\left(1.0 \geq \max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4 \cdot in$ Wall Thickness $w_{plate} = 16 \cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \end{pmatrix} \quad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix} \cdot in \quad b_{eff.5d.4} := b_{eff.5d_0} \quad b_{eff.5d.5} := b_{eff.5d_1} \quad b_{eff.5d.6} := b_{eff.5d_2}$$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$ $a_{5d4} = 11.7\,ft$ $b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$ $b_{5d.4} = 8.3\,ft$ For Wall Panel Height of

$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$ $a\_b_{5d.4} = 1.4$ $l_{c.5d.4} := H_{wall_0} - t_{floor}$ $l_{c.5d.4} = 8.3\,ft$ $H_{wall_0} = 9\,ft$

$a_{5d5} := W_{ew}$ $a_{5d5} = 11.7\,ft$ $b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$ $b_{5d.5} = 9.3\,ft$ For Wall Panel Height of

$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$ $a\_b_{5d.4} = 1.4$ $l_{c.5d.5} := H_{wall_1} - t_{floor}$ $l_{c.5d.5} = 9.3\,ft$ $H_{wall_1} = 10\,ft$

$a_{5d6} := W_{ew}$ $a_{5d4} = 11.7\,ft$ $b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$ $b_{5d.6} = 10\,ft$ For Wall Panel Height of

$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$ $a\_b_{5d.6} = 1.2$ $l_{c.5d.6} := H_{wall_2} - t_{floor}$ $l_{c.5d.6} = 10\,ft$ $H_{wall_2} = 10.7ft$

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \quad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft \quad P_{5.5} := \max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right) \quad P_{5.5} = 53.3 \cdot psf$$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4} \cdot P_{5.5} \cdot \left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 15.08 \cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6} \cdot 1\,ft$ $S_{5d.4} = 32 \cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4} \cdot S_{5d.4}$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

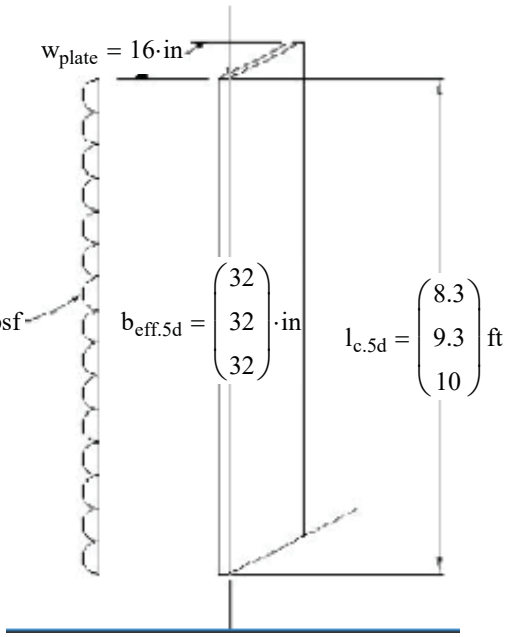
Assume only (1) end wall for resistance to lateral load $P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}{}^T = (11.7 \quad 12.4 \quad 13.1 \quad 13.8) \cdot kip$ $P_{5d.4max} := \max\left(2 \cdot V_{u.Wall.PLh0}\right)$ $P_{5d.4max} = 13.8 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}{}^T = (5.9 \quad 6.2 \quad 6.6 \quad 6.9) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \frac{\beta_{5d.5} \cdot P_{5.5} \cdot \left(l_{c.5d.5}\right)^2}{\left(t_w\right)^2} = 18.92 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \frac{\beta_{5d.6} \cdot P_{5.5} \cdot \left(l_{c.5d.6}\right)^2}{\left(t_w\right)^2} = 17.33 \cdot ksf$



$P_{5.5} = 53.3 \cdot psf$ $b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix} \cdot in$ $l_{c.5d} = \begin{pmatrix} 8.3 \\ 9.3 \\ 10 \end{pmatrix} ft$

$S_{5d.5} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15.4 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$

$P_{5d.5}^T = (13 \quad 13.8 \quad 14.6 \quad 15.4) \cdot kip$

$V_{u.Floor.PLh1}^T = (6.5 \quad 6.9 \quad 7.3 \quad 7.7) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^T = (13.9 \quad 14.9 \quad 15.8 \quad 16.8) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.8 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^T = (7 \quad 7.4 \quad 7.9 \quad 8.4) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} = 230.8 \cdot kip \cdot in$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^T = (11.4 \quad 12.1 \quad 12.8 \quad 13.5) \cdot \%$

$chk_{5d.3} := if(1.0 \geq 2 \max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} = 287.3 \cdot kip \cdot in$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

$chk_{5d.4} := if\left(P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.5} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.5} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.5}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9 \cdot kip$

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^T = (15.2 \quad 16.1 \quad 17 \quad 17.9) \cdot \%$

$chk_{5d.5} := if(1.0 \geq 2 \max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

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$P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} = 335.7 \cdot kip \cdot in$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$ $chk_{5d.6} := if\left(P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.6} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.6}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7 \cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^T = (18.9 \quad 20.2 \quad 21.5 \quad 22.8) \cdot \%$

$chk_{5d.7} := if\left(1.0 \geq 2 \max\left(Unity_{5d.4}\right), "OK", "NG"\right)$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w \cdot \rho_{min.5e.1}$ $Amin_{5e.1} = 0.096 \cdot \frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^T = (1.6 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_0} = 9\ ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^T = (2.4 \quad 2.3 \quad 2.3 \quad 2.3) \cdot kip$

$V_{L.ewh0}^T = (1.2 \quad 1.3 \quad 1.4 \quad 1.5) \cdot kip$

$Unity_{5e.1.h0}^T = (52 \quad 55.9 \quad 59.9 \quad 64) \cdot \%$ $chk_{5e.1} := if\left(1.0 \geq \max\left(Unity_{5e.1.h0}\right), "OK", "NG"\right)$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^T = (1.6 \quad 1.6 \quad 1.5 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_1} = 10\ ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^T = (2.3 \quad 2.3 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh1}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h1}^T = (57.2 \quad 61.6 \quad 66.1 \quad 70.8) \cdot \%$ $chk_{5e.2} := if\left(1.0 \geq \max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^T = (1.6 \quad 1.5 \quad 1.5 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_2} = 10.7\ ft$ $Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^T = (2.3 \quad 2.2 \quad 2.2 \quad 2.1) \cdot kip$

$V_{L.ewh2}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^T = (59 \quad 64.3 \quad 69.8 \quad 75.4) \cdot \%$ $chk_{5e.3} := if\left(1.0 \geq \max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^T = (11.7 \quad 12.4 \quad 13.1 \quad 13.8) \cdot kip$

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$Mu_{5d.4} = 3.35 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \quad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.4} = 15.7 \quad d_{5e.4} := \frac{t_w}{2} \quad d_{5e.4} = 2 \cdot in \quad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'^{T}_{5e.4} = (0.29 \quad 0.27 \quad 0.26 \quad 0.24) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \quad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right] \right]$$

$P_{d.5e.4}^{T} = (270.8 \quad 273.4 \quad 275.7 \quad 277.7) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \quad Unity_{5e.4}^{T} = (4.3 \quad 4.5 \quad 4.8 \quad 5) \cdot \%$

$chk_{5e.4} := if(1.0 \geq \max(Unity_{5e.4}), "OK", "NG")$ $chk_{5e.4} = "OK"$

$P_{5d.5}^{T} = (13 \quad 13.8 \quad 14.6 \quad 15.4) \cdot kip$

$Mu_{5d.5} = 4.2 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \quad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.5} = 15.7 \quad d_{5e.5} := \frac{t_w}{2} \quad d_{5e.5} = 2 \cdot in \quad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'^{T}_{5e.5} = (0.32 \quad 0.3 \quad 0.29 \quad 0.27) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \quad x_{5e.5} = 159.1 \cdot kip$

$$P_{d.5e.5} := x_{5e.5} \cdot \left[\left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right] \right]$$

$P_{d.5e.5}^{T} = (265 \quad 267.9 \quad 270.5 \quad 272.8) \cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}} \quad Unity_{5e.5}^{T} = (4.9 \quad 5.2 \quad 5.4 \quad 5.6) \cdot \%$

$chk_{5e.5} := if(1.0 \geq \max(Unity_{5e.5}), "OK", "NG")$ $chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (13.9 \quad 14.9 \quad 15.8 \quad 16.8) \cdot kip$

$Mu_{5d.6} = 3.85 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.6} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.6}} \quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.6} = 15.7 \quad d_{5e.6} := \frac{t_w}{2} \quad d_{5e.6} = 2 \cdot in \quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$e'_{5e.6}{}^{T} = (0.28 \quad 0.26 \quad 0.24 \quad 0.23)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P_{d.5e.6}{}^{T} = (272.3 \quad 275.1 \quad 277.5 \quad 279.7)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity_{5e.6}{}^{T} = (5.1 \quad 5.4 \quad 5.7 \quad 6)\cdot\%$

$chk_{5e.6} := if\left(1.0 \geq \max\left(Unity_{5e.6}\right), "OK", "NG"\right)$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V_{u.Floor.PLh0}{}^{T} = (5.9 \quad 6.2 \quad 6.6 \quad 6.9)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix}794.3\\833.6\\859.8\end{pmatrix}\cdot plf$

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix}714.8\\750.2\\773.8\end{pmatrix}\cdot plf$

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Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx}{}^{T} = (794.3 \quad 833.6 \quad 859.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w_{u.5f.1.DL.mx}{}^{T} = (714.8 \quad 750.2 \quad 773.8)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix}752.3\\791.6\\817.8\end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{(H_{wall} - t_{floor})}{3}$

$w_{5f.1} = \begin{pmatrix}2.8\\3.1\\3.3\end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix}w_{DL.5f.1.mx}\\-w_{DL.5f.1.mx}\end{pmatrix}}{No_{plates}} = \begin{bmatrix}\begin{pmatrix}794.25\\833.58\\859.81\end{pmatrix}\\\begin{pmatrix}-794.25\\-833.58\\-859.81\end{pmatrix}\end{bmatrix}\cdot plf$

$w_{up} := \frac{\left|P_{wind_3}\right|\cdot W_{ew}}{2}$ $\quad w_{up} = 311.9\cdot plf$ $\quad w_{u.up} := w_{up}$ $\quad w_{u.up} = 311.9\cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} + Embed_{5f.1}$ $\quad P_{5f.2.h0} = (8.7 \quad 9 \quad 9.4 \quad 9.7)\cdot kip$

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$$P_{5f.2.h1} := w_{DL.5f.1.mx_1}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} + Embed_{5f.1} \qquad P_{5f.2.h1} = (8.9 \;\; 9.3 \;\; 9.6 \;\; 10)\cdot kip$$

$$P_{5f.2.h2} := w_{DL.5f.1.mx_2}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} + Embed_{5f.1} \qquad P_{5f.2.h2} = (9.1 \;\; 9.4 \;\; 9.8 \;\; 10.1)\cdot kip$$

Tension Side

$$P_{5f.3.h0} := w_{DL.5f.1.mx_0}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} - Embed_{5f.1} \qquad P_{5f.3.h0} = (-3 \;\; -3.4 \;\; -3.8 \;\; -4.1)\cdot kip$$

$$P_{5f.3.h1} := w_{DL.5f.1.mx_1}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} - Embed_{5f.1} \qquad P_{5f.3.h1} = (-2.8 \;\; -3.2 \;\; -3.5 \;\; -3.9)\cdot kip$$

$$P_{5f.3.h2} := w_{DL.5f.1.mx_2}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} - Embed_{5f.1} \qquad P_{5f.3.h2} = (-2.7 \;\; -3 \;\; -3.4 \;\; -3.7)\cdot kip$$

$$W_{u.5f.1} := \left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| = 53.3\cdot psf \qquad Mu_{5f.1} := \frac{W_{u.5f.1}\cdot\left(H_E - 8in\right)^2}{8} = \begin{pmatrix}6.4\\8\\9.1\end{pmatrix}\cdot\frac{kip\cdot in}{ft}$$

$$d_{5.4} := 0.5\cdot t_w \qquad d_{5.4} = 2\cdot in \qquad x_{5f.2} := 0.00349 \text{ Initial Guess for reinforcement ratio}$$

$$f\left(x_{5f.2}\right) := \frac{\dfrac{\max\left(Mu_{5f.1}\right)}{\left(\dfrac{kip\cdot ft}{ft}\right)}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left(\dfrac{d_{5.4}}{in}\right)^2\cdot\left(1 - 8.85\cdot x_{5f.2}\right)} - x_{5f.2} \qquad \rho_{5f.2} := \frac{root\left(f\left(x_{5f.2}\right), x_{5f.2}\right)}{ft} \qquad \rho_{5f.2} = 0.00361\cdot\frac{1}{ft}$$

Reinforcement Ratio Required for out of plane Wind

$$A_{s.req1.5f.2.} := \rho_{5f.2}\cdot 12in\cdot d_{5.4} \qquad A_{s.req1.5f.2.} = 0.087\cdot\frac{in^2}{ft}$$

Area of Reinforcement Required for out of plane Wind

$$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9\cdot f_y\cdot\dfrac{W_{ew}}{2}}\right)$$

$$A_{s.req2.5f.2} = 0.01305\cdot\frac{in^2}{ft}$$

Area of Reinforcement Required for Tension Force from Uplift

$$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2} \qquad A_{req.5f.2} = 0.1\cdot\frac{in^2}{ft}$$

Total Area of Reinforcement Required

$$A_{s.4a} = 0.12\cdot\frac{in^2}{ft}$$

Area of Steel Provided

$$chk_{5.8} := \text{if}\left(A_{req.5f.2} \le A_{s.4a}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{5.8} = \text{"OK"}$$

Check P-Δ Effects

$$E_{c.spec} = 2991\cdot ksi \qquad I_{g.5g.1} := \frac{1}{12}\cdot 12in\cdot\left(t_w\right)^3 \qquad I_{g.5g.1} = 64\cdot in^4$$

$$P_{cr.5g.1} := \frac{\pi^2\cdot E_{c.spec}\cdot I_{g.5g.1}}{H_E^2} \qquad P_{cr.5g.1} = \begin{pmatrix}141\\115.8\\102.5\end{pmatrix}\cdot kip \qquad Ma_{g.2} := \frac{\left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right|\cdot\left(H_{wall}\right)^2}{8}\cdot 12in \qquad Ma_{g.2} = \begin{pmatrix}539.3\\665.9\\757.6\end{pmatrix}\cdot lb\cdot ft$$

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mb_{5g.2} := Ma_{g.2} \quad Mb_{5g.2} = \begin{pmatrix} 0.5 \\ 0.7 \\ 0.8 \end{pmatrix} \cdot kip \cdot ft \qquad C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}} \qquad C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}} \qquad \delta_{ns.5.6} = \begin{pmatrix} 1.2 \\ 1.2 \\ 1.2 \end{pmatrix}$$

$$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix} \qquad Ma_{g.3} = \begin{pmatrix} 629.7 \\ 806.9 \\ 944 \end{pmatrix} \cdot lb \cdot ft \qquad f_r = 427.6 \cdot psi \qquad M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}} \qquad M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking\ Occurs", "Cracking\ Doesn't\ Occur"\right)$ $\qquad chk_{5.9} = "Cracking\ Doesn't\ Occur"$

$$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}} \qquad \Delta_{5g.1} = \begin{pmatrix} 0.18 \\ 0.24 \\ 0.28 \end{pmatrix} \cdot in \qquad Mu_{5.5} := Mu_{5f.1} \cdot ft \qquad Mu_{5.5} = \begin{pmatrix} 6.4 \\ 8 \\ 9.1 \end{pmatrix} \cdot kip \cdot in$$

$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$

$Mu_{5.6} = (11.1 \quad 11.2 \quad 11.3 \quad 11.4) \cdot kip \cdot in$

$P_{5.8} := Embed_{5f.1}$

$P_{5.8} = (5.9 \quad 6.2 \quad 6.6 \quad 6.9) \cdot kip$

$$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}} \qquad e`_{5.6} = (1.9 \quad 1.8 \quad 1.7 \quad 1.7) \cdot in$$

$$u_{5.} := \frac{f_y}{0.85 \cdot f`_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f`_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

$$\rho_{5.8} := \frac{A_{s.4a}}{\max\left(w_{5f.1}\right) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}} + \sqrt{\left(1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max\left(e`_{5.6}\right) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 229.3 \\ 256.9 \\ 275.2 \end{pmatrix} \cdot kip$$

$chk_{5.10} := if\left(\max\left(P_{5.8}\right) \le \min\left(\phi P_{5.6}\right), "OK", "NG"\right)$ $\qquad chk_{5.10} = "OK"$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 37.6 \\ 37.5 \\ 37.4 \\ 37.3 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 30 ' "} \\ \text{"Min Flr Uniform Weight, for L = 32 ' "} \\ \text{"Min Flr Uniform Weight, for L = 34 ' "} \\ \text{"Min Flr Uniform Weight, for L = 36 ' "} \end{pmatrix}$$

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$$wg_{f.mx} = \begin{pmatrix} 41.6 \\ 41.5 \\ 41.4 \\ 41.3 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$$ Wall Weight

$$w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf$$ Wall Misc. Weight

$DL_{r.mx} = 73.4 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 66.4 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -53.48 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 46 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.145$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

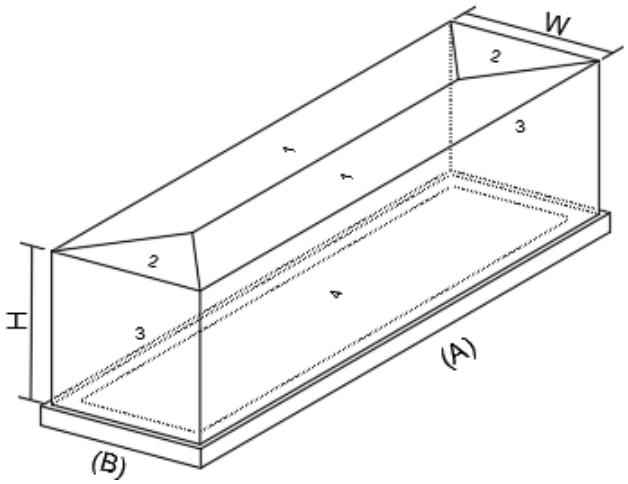
$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.116$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (364 \quad 388 \quad 412 \quad 436)\ ft^2$

$A_f^T = (320.99 \quad 342.9 \quad 364.82 \quad 386.74)\ ft^2$

$L_{wall}^T = (81.92 \quad 85.92 \quad 89.92 \quad 93.92)\ ft$



$$DL_{A.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) + \frac{W_f}{2} \cdot (wgt_{flr}) \ldots + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

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$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right)\ \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 65.49\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right)\ \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 58.25\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor}\ \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\ \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}\right] \qquad \begin{aligned} z_{mx} &= 67.88\cdot in \\ z_{mx} &= 5.66\cdot ft\end{aligned}$$

$ftg_{exp} := 36in$ Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.81\ ft$ Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\ ft$ Moment Arm for Floor Level

$DL_{A.f0.mx}^{T} = (1147.8 \quad 1147.3 \quad 1146.8 \quad 1146.5)\cdot plf$

$DL_{A.f0.mn}^{T} = (1099 \quad 1089.5 \quad 1081.2 \quad 1073.8)\cdot plf$

$DL_{B.f0.mx}^{T} = (854.3 \quad 853.8 \quad 853.3 \quad 853)\cdot plf$

$DL_{B.f0.mn}^{T} = (833.5 \quad 824 \quad 815.7 \quad 808.3)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1640 \quad 1640 \quad 1639 \quad 1639)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (1099 \quad 1090 \quad 1081 \quad 1074)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (867 \quad 866 \quad 866 \quad 866)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (606 \quad 605 \quad 605 \quad 605)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C}$ $\qquad E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.com

$E_{O.A.E.f0.C}^{T} = (219.9 \;\; 219.9 \;\; 219.8 \;\; 219.7)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-219.9 \;\; -219.9 \;\; -219.8 \;\; -219.7)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (43.5 \;\; 40.7 \;\; 38.2 \;\; 36.1)\cdot plf$

$E_{O.B.E.f0.T}^{T} = (-43.5 \;\; -40.7 \;\; -38.2 \;\; -36.1)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (210.7 \;\; 210.7 \;\; 210.6 \;\; 210.6)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-210.7 \;\; -210.7 \;\; -210.6 \;\; -210.6)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (42.3 \;\; 39.6 \;\; 37.2 \;\; 35.1)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-42.3 \;\; -39.6 \;\; -37.2 \;\; -35.1)\cdot plf$

$E_{O.A.E.f2.C}^{T} = (201.6 \;\; 201.5 \;\; 201.4 \;\; 201.4)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-201.6 \;\; -201.5 \;\; -201.4 \;\; -201.4)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (41.2 \;\; 38.5 \;\; 36.2 \;\; 34.2)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-41.2 \;\; -38.5 \;\; -36.2 \;\; -34.2)\cdot plf$

$$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$$

$$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$$

$$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$$

$$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$$

$E_{8p.A.f0}^{T} = (647 \;\; 646.8 \;\; 646.7 \;\; 646.5)\cdot plf$

$E_{8n.A.f0}^{T} = (-188.1 \;\; -188.3 \;\; -188.5 \;\; -188.7)\cdot plf$

$E_{8p.B.f0}^{T} = (269.1 \;\; 266.3 \;\; 263.8 \;\; 261.6)\cdot plf$

$E_{8n.B.f0}^{T} = (-25.9 \;\; -23.2 \;\; -20.7 \;\; -18.6)\cdot plf$

$E_{8p.A.f1}^{T} = (620 \;\; 619.8 \;\; 619.6 \;\; 619.5)\cdot plf$

$E_{8p.B.f1}^{T} = (262 \;\; 259.3 \;\; 256.9 \;\; 254.7)\cdot plf$

$E_{8n.A.f1}^{T} = (-167.1 \;\; -167.1 \;\; -167 \;\; -167)\cdot plf$

$E_{8n.B.f1}^{T} = (-18.5 \;\; -15.8 \;\; -13.4 \;\; -11.3)\cdot plf$

$E_{8p.B.f2}^{T} = (255 \;\; 252.3 \;\; 249.9 \;\; 247.8)\cdot plf$

$E_{8n.A.f2}^{T} = (-159.9 \;\; -159.9 \;\; -159.8 \;\; -159.8)\cdot plf$

$E_{8p.A.f2}^{T} = (593 \;\; 592.8 \;\; 592.6 \;\; 592.5)\cdot plf$

$E_{8n.B.f2}^{T} = (-18 \;\; -15.4 \;\; -13.1 \;\; -11)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$LL_{r.A.} := \frac{W_r}{2}\cdot(w_{llr})$ $LL_{r.A.} = 600\cdot plf$ $LL_{r.B.} := \frac{W_r}{6}\cdot(w_{llr})$ $LL_{r.B.} = 200\cdot plf$

$LL_{A.f0} := \frac{W_f}{2}\cdot(w_{llf.0})$ $LL_{A.f0} = 1369.79\cdot plf$ $LL_{B.f0} := s_{t.rib}\cdot(w_{llf.0})$ $LL_{B.f0} = 500\cdot plf$

$LL_{A.f1} := \frac{W_f}{2}\cdot(w_{llf.1})$ $LL_{A.f1} = 1095.83\cdot plf$ $LL_{B.f1} := s_{t.rib}\cdot(w_{llf.1})$ $LL_{B.f1} = 400\cdot plf$

$LL_{A.f2} := \frac{W_f}{2}\cdot(w_{llf.2})$ $LL_{A.f2} = 821.88\cdot plf$ $LL_{B.f2} := s_{t.rib}\cdot(w_{llf.2})$ $LL_{B.f2} = 300\cdot plf$

$SL_A := \frac{W_r}{2}\cdot(p_f)$ $SL_A = 190.51\cdot plf$ $SL_B := \frac{W_r}{6}\cdot(p_f)$ $SL_B = 63.5\cdot plf$

$W_{up.A} := \frac{W_r}{2}\cdot P_{wind_3}$ $W_{up.A} = -320.86\cdot plf$ $W_{up.B} := \frac{W_r}{6}\cdot P_{wind_3}$ $W_{up.B} = -106.95\cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $W_{OT.A.C} = 618.5\cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $W_{OT.B.C} = 231.4\cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $W_{OT.A.T} = -618.5\cdot plf$ $W_{OT.B.T} := -W_{OT.B.C}$ $W_{OT.B.T} = -231.4\cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $W_{A.mx} = 297.6\cdot plf$ $W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $W_{B.mx} = 124.4\cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $W_{A.mn} = -939.4\cdot plf$ $W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $W_{B.mn} = -338.3\cdot plf$

b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr} = 100\cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.0} = 250\cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \qquad Load_{f0} = \begin{pmatrix} 1147.76 \\ 1073.76 \\ 1369.79 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 647.03 \\ -188.67 \end{pmatrix} \cdot plf$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f0} := asd \cdot Load_{f0} \quad LC_{8.f0} = \begin{pmatrix} 2518 \\ 1748 \\ 2775 \\ 1326 \\ 584 \\ 2698 \\ 1771 \\ 823 \\ 81 \\ 1601 \\ 1016 \\ 2658 \\ 2219 \\ 1097 \\ 512 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 13.32 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 14 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f0} := \max(15in, w_{8.f0.min})$ $\quad Fdn_{W.f0} = 15 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 15 \cdot in$

consider $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \quad Load_{f1} = \begin{pmatrix} 1147.76 \\ 1073.76 \\ 1095.83 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 620.01 \\ -167.11 \end{pmatrix} \cdot plf$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

$$\text{ASD} = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f1} := asd \cdot Load_{f1} \quad LC_{8.f1} = \begin{pmatrix} 2244 \\ 1748 \\ 2570 \\ 1326 \\ 584 \\ 2493 \\ 1565 \\ 823 \\ 81 \\ 1582 \\ 1031 \\ 2438 \\ 2025 \\ 1078 \\ 527 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 12.33 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 13 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f1} := \max(14in, w_{8.f1.min})$ $\quad Fdn_{W.f1} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

consider $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \quad Load_{f2} = \begin{pmatrix} 1147.76 \\ 1073.76 \\ 821.88 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 592.99 \\ -159.91 \end{pmatrix} \cdot plf$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f2} := asd \cdot Load_{f2} \quad LC_{8.f2} = \begin{pmatrix} 1970 \\ 1748 \\ 2364 \\ 1326 \\ 584 \\ 2287 \\ 1360 \\ 823 \\ 81 \\ 1563 \\ 1036 \\ 2218 \\ 1823 \\ 1059 \\ 532 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 11.35 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 12 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f2} := \max(14in, w_{8.f2.min})$ $\quad Fdn_{W.f2} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &
$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 14 \cdot in$

c. Slab on Grade Foundation

1.) Criteria: use slab on grade. $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $\quad Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.long} \qquad A_{s.long.prov} = 0.39 \cdot in^2$$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if\left(A_{s.long.prov} \geq A_{s.long}, "OK", "NG"\right)$ $chk_{8.1} = "OK"$

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp} \qquad A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4 \qquad Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.temp}$$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if\left(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG"\right)$ $chk_{8.2} = "OK"$

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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 66.375 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 73.4 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 100 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 46 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 35.8 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 32.3 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 53.5 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 3.03 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 3.035 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 9.364 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.021 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.021 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.401 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.401 \cdot klf$

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SEISMIC LOAD PARAMETERS:

For worst case seismic load in E-W direction, look at longest, widest shelter.

Seismic Coefficient

$C_s = 0.145$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 42.89\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 39.84\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 34.82\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 29.38\cdot kip$

Total Seismic Shear at Roof

$$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$$ $V_{smaxW\_rf} = 8.723\cdot kip$

$$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$$ $V_{sminW\_rf} = 7.888\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$$ $v_{EW\_smaxW} = 0.145\cdot klf$

$$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$$ $v_{EW\_sminW} = 0.11\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$$ $v_{NS\_smaxW} = 0.374\cdot klf$

$$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$$ $v_{NS\_sminW} = 0.338\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$$ Modification Factor for Eccentric Loading

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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k_c Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix} =$ ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.035 \cdot klf$ Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.374 \cdot klf$ Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.16 \cdot klf$ Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.241 \cdot klf$ Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.094 \cdot klf$ Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.401 \cdot klf$ Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

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Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 8$

$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1}$ $s_{REW} = 16\cdot in$

$s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$ $s_{EWR} = 16.57\cdot in$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, \min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, \min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$ $Vu_{o.s.REW} = 0.016\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$ $Vu_{i.s.REW} = 0.582\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$ $Vu_{o.w.REW} = 0.146\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$ $Vu_{i.w.REW} = 0.624\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.REW} := F_{7.1}\cdot\max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$

$Tu_{T.REW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$ $Vu_{o.s.EWR} = 0.017\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$ $Vu_{i.s.EWR} = 0.602\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.EWR} := F_{7.2}\cdot\max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$ $Vu_{o.w.EWR} = 0.152\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$ $Vu_{i.w.EWR} = 0.647\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$ $Tu_{T.EWR} = 0.152\cdot kip$ 0.9DL + 1.0W T, roof uplift

Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 6\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

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Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 6 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 135 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 6.86 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 6 \cdot in$ $h_{min.7.1a} = 6 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 161 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 4.51 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 5.145 \\ 3.38 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min\left(\left(0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1}\right)\right)$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.175 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.213 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.022$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.255 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.315 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.REW.1} = 0.177 \cdot in$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.REW.2} = 0.2 \cdot in$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2}$$ $l_{w\_Rqrd.REW.3} = 2.5 \cdot in$

$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right)$ $l_{w\_reqd.REW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.035\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.145\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.321\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.44\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.188\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.021\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 36\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 16\cdot in$ $s_{EWR} = 16.57\cdot in$

$s_{7.3} := if(s_{L.7.3} = 0in, s_{w.7.3}, if(s_{w.7.3} = 0in, s_{L.7.3}, min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := if(s_{L.7.4} = 0in, s_{w.7.4}, if(s_{w.7.4} = 0in, s_{L.7.4}, min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.122\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.153\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 0.658\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.074\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW})\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.122\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.153\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 0.658\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.074\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW})\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.104 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.245 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.101 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.231 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

MODULAR CONNECTIONS, LLC.
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Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.057\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.194\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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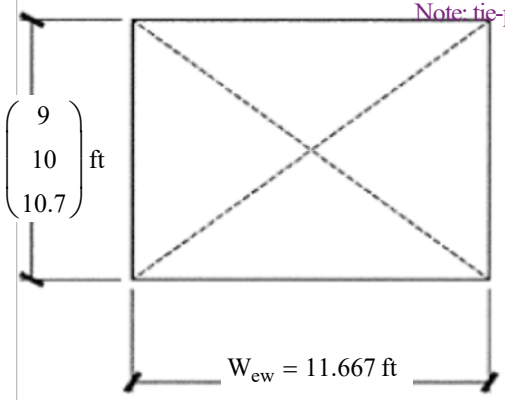
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 8$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}$ ft

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (11.714 \quad 13.429 \quad 14.571$ $s_{SWEW}^{T} = (11.714 \quad 13.429 \quad 14.571)$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6$ ft Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 5.275\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{ew}\cdot L_{ew}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (0.498 \quad 0.467 \quad 0.44 \quad 0.416)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.007\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 0.816\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10$ ft

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

$M_{ot\_w.EWSW}^{T} = (59.9 \quad 62.8 \quad 65.8 \quad 68.8)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[(DL_{r.mx} + 0.25\cdot w_{llr})\cdot W_r\cdot L_r + \frac{\max\left[(\omega_{wall})\cdot\left[\max(H_{wall})\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (77 \quad 80.4 \quad 83.8 \quad 87.2)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (95.2 \quad 100.5 \quad 105.8 \quad 111)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (-25.81 \quad -27.58 \quad -29.35 \quad -31.12)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (-18.2 \quad -20.07 \quad -21.94 \quad -23.82)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net})$ $M_{ot.EWSW} = 0\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := if(s_{L.7.5} = 0in, s_{w.7.5}, if(s_{w.7.5} = 0in, s_{L.7.5}, min(s_{L.7.5}, s_{w.7.5}))) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := if(s_{L.7.6} = 0in, s_{w.7.6}, if(s_{w.7.6} = 0in, s_{L.7.6}, min(s_{L.7.6}, s_{w.7.6}))) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.293\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.293\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wal (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.293 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.293 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.068$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.1$ < 1.0

$Result_{EWSW} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW} =$ "not required" < 1.2

$chk_{6.EWSW} =$ "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.069$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.181$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.121 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.121 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right)$ $l_{w\_reqd.EWSW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.71 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.309 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.017 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.374 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.401 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.165 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.094 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\,ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$M_{ot.s.FEW}^{T} = (72.8 \quad 77.6 \quad 82.4 \quad 87.2)\cdot kip\cdot ft$

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix}0.5\cdot(L_0)^2\\0.5\cdot(L_1)^2\\0.5\cdot(L_2)^2\\0.5\cdot(L_3)^2\end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$$

OT Moment, resisting

$M_{res.FEW}^{T} = (1038.2 \quad 1107.4 \quad 1176.7 \quad 1245.9)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-534.6 \quad -546.6 \quad -555.3 \quad -560.9)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = (-965.4 \quad -1\times10^{3} \quad -1.1\times10^{3} \quad -1.2\times10^{3})\cdot kip\cdot ft$

$M_{ot.FEW} := \max(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net})$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right] \qquad v_{ot.w.FEW} = 0\cdot kip$$

$$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right) \qquad v_{ot.s.FEW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor $\quad$ $Plate_{7.8}$ = "#2" at inside face of end wall

No. of Plates & maximum plate spacing $\quad No_{pl.FEW} = 8$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \qquad s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1}$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$s_{F\_EW} = 13.64\cdot in$ $\quad s_{EW\_F} = 14.86\cdot in$

$s_{7.7} := \text{if}(s_{L.7.7} = 0in, s_{w.7.7}, \text{if}(s_{w.7.7} = 0in, s_{L.7.7}, \min(s_{L.7.7}, s_{w.7.7}))) = 3\cdot in$ $\quad s_{7.7} = 3\cdot in$

$$F_{7.7} := 0.5 + \frac{0.5s_{7.7} + 2e_{PL.F\_EW}}{s_{7.7}} \qquad F_{7.7} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.7

$s_{7.8} := \text{if}(s_{L.7.8} = 0in, s_{w.7.8}, \text{if}(s_{w.7.8} = 0in, s_{L.7.8}, \min(s_{L.7.8}, s_{w.7.8}))) = 3\cdot in$ $\quad s_{7.7} = 3\cdot in$

$$F_{7.8} := 0.5 + \frac{0.5s_{7.8} + 2e_{PL.F\_EW}}{s_{7.8}} \qquad F_{7.8} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.8

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right) \qquad Vu_{i.s.FEW} = 0.921\cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{i.w.FEW} = 0.683\cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$$Tu_{po.s1.FEW} := F_{7.7}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right) \qquad Tu_{po.s1.FEW} = 0.01\cdot kip$$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Tu_{po.w.FEW} = 0.32\cdot kip$$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{o.9dl.FEW} = 1.087\cdot kip$$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7}\cdot\frac{v_{ot.w.FEW}}{No_{pl.FEW}} \qquad V_{ot.w.FEW} = 0\cdot kip$$ 1.0W

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot.s.FEW} := F_{7.7}\cdot\frac{v_{ot.s.FEW}}{No_{pl.FEW}} \qquad V_{ot.s.FEW} = 0\cdot kip$$ 1.0E

$$Vu_{o.wup.FEW} := F_{7.7}\cdot\max\left[\left(1.0v_{up.w.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0\cdot kip\right]$$

$$Vu_{o.wup.FEW} = 0\cdot kip$$ 0.9D+1.0W

$$Vu_{o.sup.FEW} := F_{7.7}\cdot\max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0\cdot kip\right)$$ Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.sup.FEW} = 0\cdot kip$$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7}\cdot\left[1.2\left(v_{DL.FEW}\right) + 1.6v_{LL.FEW}\right]\cdot\frac{W_{ew}}{No_{pl.FEW}}$$

$$Vu_{o.12D.16L.FEW} = 2.29\cdot kip$$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Endwall (Floor Wall Plates)

$$Tu_{po.w.EWF} := F_{7.8}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Tu_{po.w.EWF} = 0.32\cdot kip$$ pullout, wind suction at bot of wall and bot half of two sides of wall

$$Tu_{po.s2.EWF} := F_{7.8}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right) \qquad Tu_{po.s2.EWF} = 0.01\cdot kip$$ 0.3E pullout, seismic

$$Vu_{i.s.EWF} := F_{7.8}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right)$$

$$Vu_{i.s.EWF} = 0.921\cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w2.EWF} := F_{7.8}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{i.w2.EWF} = 0.683\cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$$Vu_{o.9dl.EWF} := F_{7.8}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{o.9dl.EWF} = 1.087\cdot kip$$ 0.9D Shear from DL only

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$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - \left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 0.988 \cdot kip$ 0.9D+1.0W

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[\left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

$Vu_{o.sdn.EWF} := F_{7.8} \cdot \left(Vu_{o.9dl.FEW} + V_{ot.s.FEW}\right)$ $Vu_{o.sdn.EWF} = 1.268 \cdot kip$ 0.9D+1.0E

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6 v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 2.29 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$

$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75 \cdot V_{cbg\_op.7.7} \\ 0.75 \cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_EW} = \min\left(\left(0.75 \cdot N_{sa.7.7} \quad 0.75 \cdot N_{cbg.7.7} \quad 0.7 \cdot N_{pn.7.7}\right)\right)$ $\phi P_{pl.F\_EW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.036 \quad \text{or} \quad \left[\frac{Vu_{o.wup.FEW}}{0.75 \cdot V_{cbg\_op.7.7}} + \frac{Vu_{i.w.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.14 < 1.0$$

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.036 \quad \text{or} \quad \left[\frac{Vu_{o.sup.FEW}}{\left(0.75 \cdot V_{cbg\_op.7.7}\right)} + \frac{Vu_{i.s.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.188 < 1.0$$

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$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.673 \quad < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted"

$IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $V_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719\cdot in$ $h_{min.7.8} = 4\cdot in$ $c_{a1\_op.7.8} = 6.5\cdot in$ $c_{a2\_op.7.8} = 10.5\cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90\cdot in^2$ $A_{Vco\_op.7.8} = 190\cdot in^2$ $e'_{V.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0\cdot in$ $s2_{7.8a} = 3\cdot in$ $l_{e.7.8} = 1.719\cdot in$ $h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$ $h_{min.7.8a} = 4\cdot in$

$c_{a1\_ip.7.8} = 13.5\cdot in$ $c_{a2\_ip.7.8} = 6.5\cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107\cdot in^2$ $A_{Vco\_ip.7.8} = 820\cdot in^2$ $e'_{V.7.8a} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3\cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74\cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8}\cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74\cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75\cdot V_{cbg\_op.7.8} \\ 0.75\cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix}\cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\begin{pmatrix} 0.75\cdot N_{sa.7.8} & 0.75\cdot N_{cbg.7.8} & 0.7\cdot N_{pn.7.8} \end{pmatrix}\right)$ $\phi P_{pl.EW\_F} = 4.305\cdot kip$

Plate Summary

$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.002$ or $\max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.779$ < 1.0

$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.074$ or $\max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.591$ < 1.0

$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75\cdot V_{cbg\_op.7.8}}\right) = 0.668$ < 1.0

$Result_{EW\_F}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F}$ = "not required" < 1.2

$chk_{6.EW\_F}$ = "OK"

Weld Between $Plate_{7.7}$ = "#1B" and $Plate_{7.8}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in}$$ $l_{w\_Rqrd.FEW.1} = 0.221\cdot in$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in}$$ $l_{w\_Rqrd.FEW.2} = 0.27\cdot in$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in}$$ $l_{w\_Rqrd.FEW.3} = 0.364\cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^{2} + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^{2} + Vu_{i.s.EWF}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.459\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.459\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$$l_{w\_reqd.EWSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $\quad w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $\quad v_{dl.FSW} = 0.924\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $\quad v_{ll.FSW} = 0.6\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $\quad t_{o.s.FSW} = 0.035\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $\quad v_{i.s.FSW} = 0.145\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $\quad v_{i.w.SW} = 0.0211\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $\quad v_{up.w.FSW} = 0.321\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $\quad t_{po.w.FSW} = 0.172\cdot klf$ Wind load suction on bottom of side wall

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^{T} = (72.8 \quad 77.6 \quad 82.4 \quad 87.2)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $\quad M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^{T} = (1038.2 \quad 1107.4 \quad 1176.7 \quad 1245.9)\cdot kip\cdot ft$

$$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.w.FSW} = 0\cdot kip$$

$$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.s.FSW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9}$ = "#1B" at top of floor

$Plate_{7.10}$ = "#2" at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 15\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$$s_{7.9} := \text{if}\left(s_{L.7.9} = 0in, s_{w.7.9}, \text{if}\left(s_{w.7.9} = 0in, s_{L.7.9}, \min\left(s_{L.7.9}, s_{w.7.9}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}} \qquad F_{7.9} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.9

$$s_{7.10} := \text{if}\left(s_{L.7.10} = 0in, s_{w.7.10}, \text{if}\left(s_{w.7.10} = 0in, s_{L.7.10}, \min\left(s_{L.7.10}, s_{w.7.10}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}} \qquad F_{7.10} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.FSW} = 0.251\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.FSW} = 0.051\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.FSW} = 0.064\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.FSW} = 0.031\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.FSW} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.FSW} = 1.039\cdot kip$ 0.9D Shear from DL only

$$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}} \qquad V_{ot\_w.FSW} = 0\cdot kip$$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}} \qquad V_{ot\_s.FSW} = 0\cdot kip$$

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\begin{bmatrix}\left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, \left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, 0\cdot kip \\ + V_{ot\_w.FSW} \qquad\qquad + -V_{ot\_w.FSW}\end{bmatrix}$$

$Vu_{o.w.FSW} = 1.68\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$$

$Vu_{o.s.FSW} = 1.212\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$$

$Vu_{o.12D.16L.FSW} = 3.017\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.251\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.051\cdot kip$ 0.3W T, pullout seismic

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$Vu_{i.s.SWF} := F_{7.10} \cdot 0.3 v_{i.s.FSW} \cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.064 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10} \cdot 1.0 v_{i.w.SW} \cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.031 \cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := (0.9 v_{dl.FSW}) \cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.039 \cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0 \cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0 \cdot kip$

$$Vu_{o.w.SWF} := F_{7.10} \cdot \max\left[\begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, \begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +-V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, 0 \cdot kip\right]$$

$Vu_{o.w.SWF} = 1.68 \cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.SWF} := F_{7.10} \cdot \max(V_{ot\_s.SWF} - 0.9 v_{dl.FSW} \cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9 v_{dl.FSW} \cdot s_{max.FSW}, 0 \cdot kip)$

$Vu_{o.s.SWF} = 1.212 \cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.SWF} := F_{7.10} \cdot [1.2(v_{dl.FSW}) + 1.6 v_{ll.FSW}] \cdot s_{max.FSW}$

$Vu_{o.12D.16L.SWF} = 3.017 \cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5 \cdot in$ $w_{pl.7.9} = 5 \cdot in$ $x_{Le.7.9} = 1 \cdot in$ $x_{we.7.9} = 4 \cdot in$ $t_{pl.7.9} = 0.313 \cdot in$

$h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0 \cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5 \cdot in$ $A_{SE.7.9} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $N_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.028 \quad \text{or} \quad \left[\frac{Vu_{o.w.FSW}}{\left(0.75 \cdot V_{cbg\_op.7.9}\right)} + \frac{Vu_{i.w.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.501 \quad < 1.0$$

$$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.006 \quad \text{or} \quad \left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.37 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.887 \quad < 1.0$$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

MODULAR CONNECTIONS, LLC.
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.058 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.501 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.012 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.379 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.876 < 1.0$$

$Result_{SWF}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWF}$ = "not required" < 1.2

$chk_{6.SWF}$ = "OK"

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.1} = 0.498 \cdot in$$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.2} = 0.356 \cdot in$$

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.498 \cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.356 \cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.149 \cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min\left(w_{pl.7.9}, w_{pl.7.10}\right)}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5 \cdot in$$

$l_{w\_reqd.FSW} := \max\left(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6}\right)$ $l_{w\_reqd.FSW} = 2.5 \cdot in$

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g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $\quad wgt_r = 66.375\cdot psf$ — Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $\quad wgt_w = 39.333\cdot psf$ — Wall Dead Load

$wgt_{flr}^{T} = (37.56 \quad 37.47 \quad 37.39 \quad 37.32)\cdot psf$ — Floor Dead Load

$W_{lat} = 45.986\cdot psf$ — Wind Load

$w_{llf.0} = 250\cdot psf$ $\quad w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200\cdot psf$ $\quad w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150\cdot psf$ $\quad w_{llf.2.ref}$ = "Floor Live Load Option 2"

$A_r := L_r\cdot W_r$ $\quad A_r^{T} = (364 \quad 388 \quad 412 \quad 436)\ ft^2$ — Area of Roof/Floor

$A_f := L_f\cdot W_f$ $\quad A_f^{T} = (321 \quad 342.9 \quad 364.8 \quad 386.7)\ ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$

Allowable Resisting Moment

$M_R^{T} = (540.6 \quad 554.3 \quad 568.1 \quad 581.8)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$

Overturning Moment due to Wind

$M_{OT.wind}^{T} = (122.3 \quad 130.4 \quad 138.6 \quad 146.7)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^{T} = (183.4 \quad 195.6 \quad 207.9 \quad 220.1)\cdot kip\cdot ft$

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\left[\begin{matrix} L_r\cdot W_r\cdot(wgt_r)\cdot\max(H_E) + L_r\cdot W_r\cdot(0.25w_{llr})\cdot\max(H_E) \ldots \\ +\left(2L_f\cdot\max(H_E) + 2W_{ew}\cdot\max(H_E)\right)\cdot\frac{wgt_w\cdot\max(H_E)}{2} \ldots \\ + L_f\cdot W_f\cdot wgt_{flr}\cdot\frac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot(0.25w_{llf.0})\cdot t_{floor} \end{matrix}\right]$$

Overturning Moment due to seismic

$M_{OT.seismic}^{T} = (62.719 \quad 66.343 \quad 69.968 \quad 73.592)\cdot kip\cdot ft$

$1.5\,M_{OT.seismic}^{T} = (94.079 \quad 99.515 \quad 104.951 \quad 110.388)\cdot kip\cdot ft$

$M_{ot.w.net} := (1.5M_{OT.wind} - M_R)$ $\quad M_{ot.s.net} := (1.5M_{OT.seismic} - M_R)$

$M_{ot.w.net}^{T} = (-357.2 \quad -358.7 \quad -360.2 \quad -361.7)\cdot kip\cdot ft$

$M_{ot.s.net}^{T} = (-446.5 \quad -454.8 \quad -463.1 \quad -471.4)\cdot kip\cdot ft$

$chk_{S7.15} := \text{if}(\max(M_{ot.w.net}, M_{ot.s.net}) \geq 0, \text{"NG"}, \text{"OK"})$ $\quad chk_{S7.15}$ = "OK"

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Sliding

$$WT := \begin{pmatrix} L_r \cdot W_r \cdot wgt_r + L_f \cdot \min(H_E) \cdot wgt_w \ldots \\ + 2W_{ew} \cdot \min(H_E) \cdot wgt_w + 2L_f \cdot W_f \cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^T = (150.083 \quad 152.435 \quad 154.787 \quad 157.138) \cdot kip$$

$$WT_{max} := \max\begin{bmatrix} A_r \cdot (wgt_r + w_{misc.r}) + A_f \cdot (wgt_{flr} + w_{misc.f}) \ldots \\ + 2L_f \cdot \max(H_E) \cdot (wgt_w + w_{misc.w}) \ldots \\ + 2 \cdot W_{ew} \cdot \max(H_E) \cdot wgt_w + [A_r \cdot w_{llr} + A_r \cdot (w_{llf.0})] \end{bmatrix}$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 289.811 \cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in \cdot 4in$ $A_{pad} = 16 \cdot in^2$

$S_{bp} := 3ft$ Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad} = 5.015 \cdot kip$ Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 313.4 \cdot psi$ Compressive Stress per Pad

$F_{pad} := 800psi$ Allowable Stress per pad

$F_P := 0.7 \cdot 0.85 f'_{c.fdn}$ $F_P = 1785.1 \cdot psi$ Allowable Stress of Fdn Concrete

$chk_{S7.16} := if(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, "OK", "NG")$ $chk_{S7.16} = "OK"$

$$wt_{pad.big} := \max\left[\frac{\max(WT_{max})}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad.big} = 9.249 \cdot kip$ Weight per Bearing Pad

$$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$$

$f_{pad.big} = 578.1 \cdot psi$ Compressive Stress per Pad

$chk_{S7.17} := if(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, "OK", "NG")$ $chk_{S7.17} = "OK"$

$\mu := 0.25$ Coefficient of Friction

Wind $V_{net.w.sl} := \max\begin{bmatrix} 0kip, \begin{bmatrix} \mu \cdot [-(WT) + 1.5 |P_{wind_3}| \cdot (A_r)] \ldots \\ + 1.5W_{lat} L \cdot \max(H_E) \end{bmatrix} \end{bmatrix}$ $V_{net.w.sl} = 0 \cdot kip$ Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := \max\begin{bmatrix} 0kip, 1.5 \cdot \begin{bmatrix} 0.7C_s \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} + -\mu \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} \end{bmatrix} \end{bmatrix}$

Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$V_{net.s.sl} = 0 \cdot kip$

$chk_{S7.18} := if(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, "Slide Plates Req'd", "OK")$ $chk_{S7.18} = "OK"$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \begin{cases} \max\left(2, ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) & \text{if } \frac{ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ \max\left[2, \left(ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] & \text{otherwise} \end{cases}$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34)\,ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3\,ft$ on center.

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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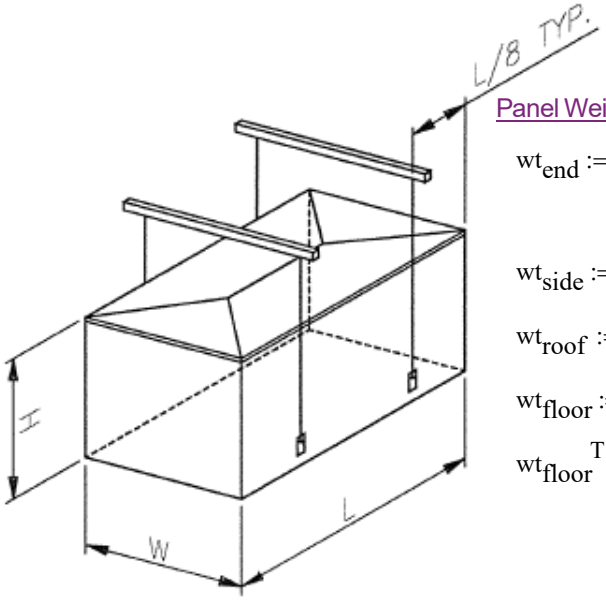
8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5191 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 445 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 797 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (438 \quad 437 \quad 436 \quad 435) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

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The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) $M/(0.9f_y d)$			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	30.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.222	0.199
12.00	32.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.257	0.226
12.00	34.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.295	0.253
12.00	36.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.336	0.280
12.00	30.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.210	0.154
12.00	32.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.244	0.175
12.00	34.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.280	0.196
12.00	36.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.318	0.216
12.00	30.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.204	0.135
12.00	32.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.236	0.153
12.00	34.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.271	0.171
12.00	36.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.309	0.190

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b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

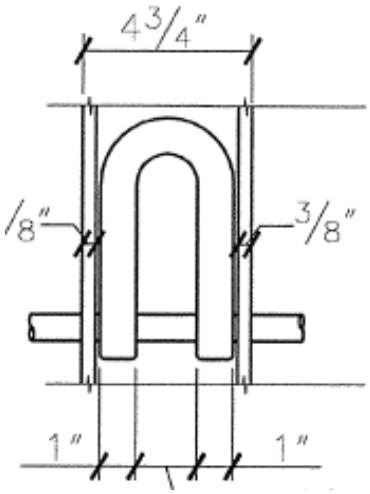
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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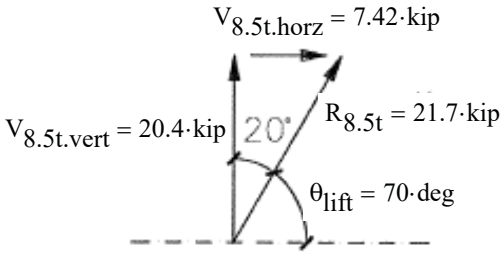


$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

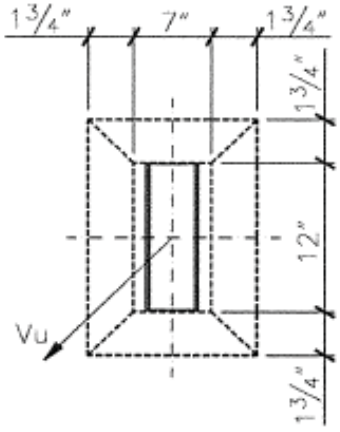
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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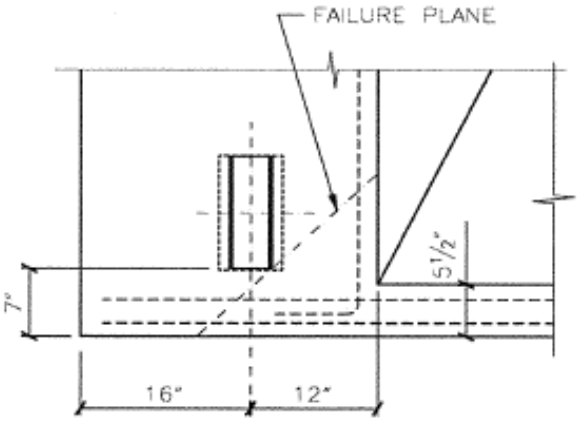
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

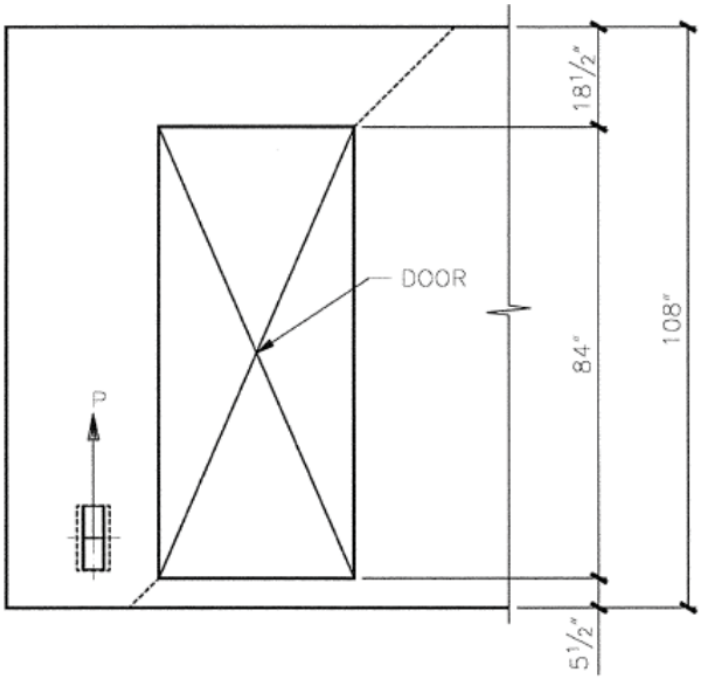
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

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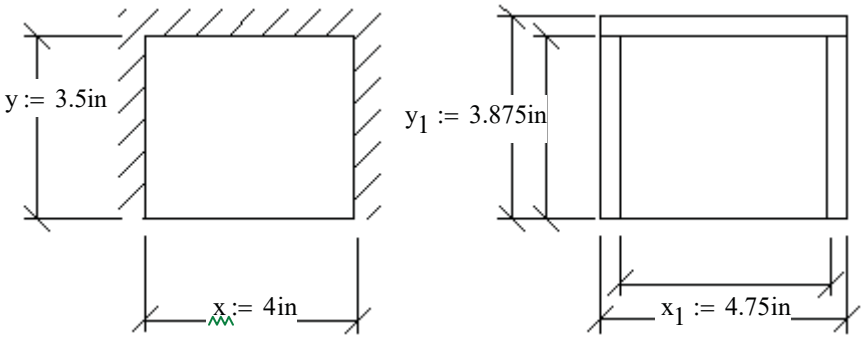
Bearing Pressure

Vertical



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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer: Modular Connections, LLC

$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

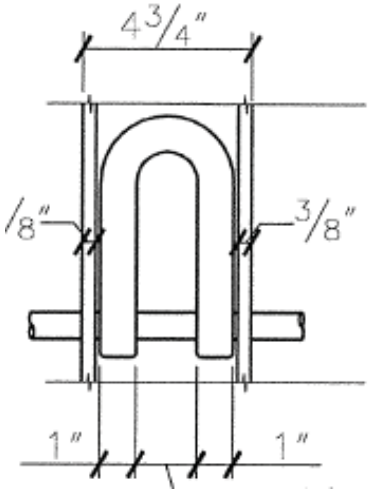
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

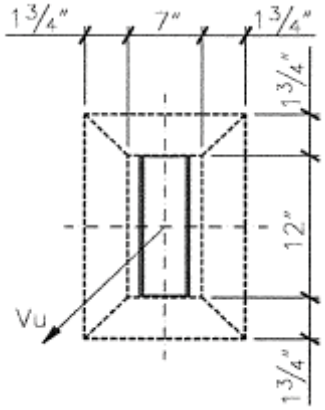
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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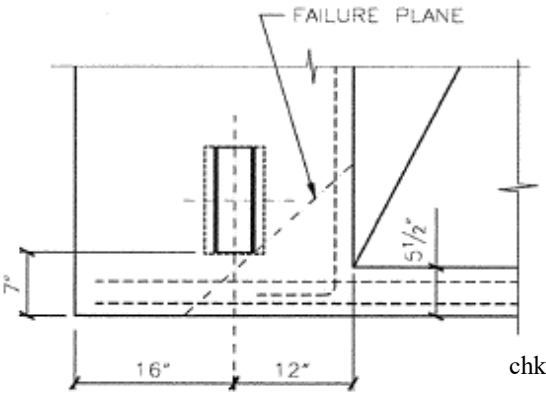
Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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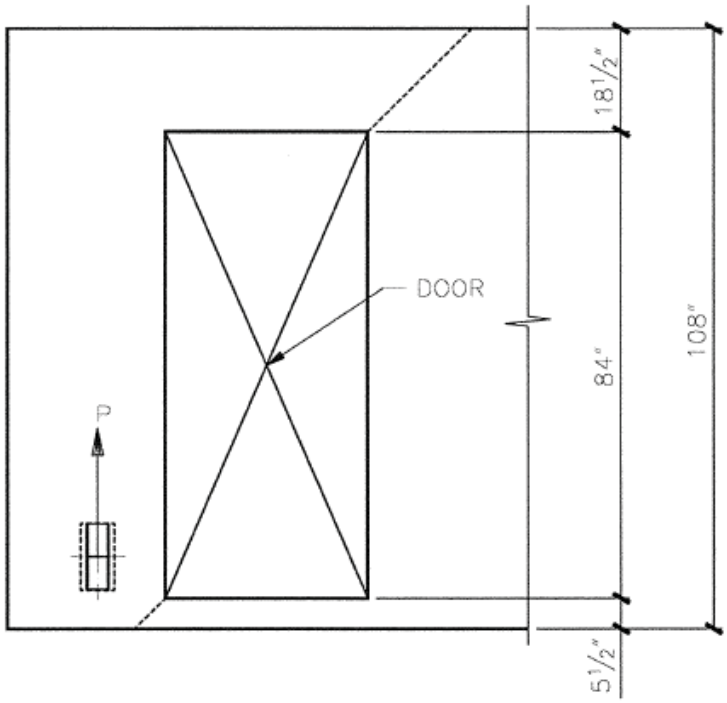


$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

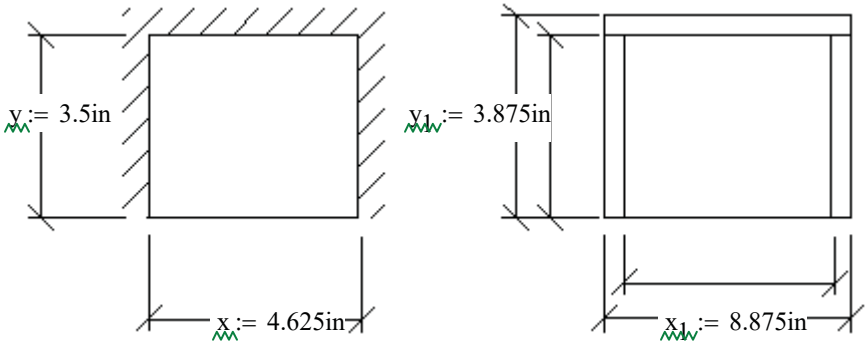
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

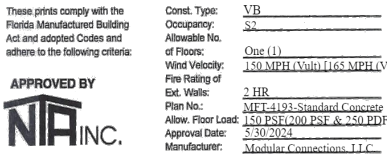
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 28.7 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 15.8 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2 \max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 32 \cdot kip$ Wall Dead Load

$w_{ew} := 2 \max(wt_{end})$ $w_{ew} = 10.4 \cdot kip$ Weight of End Wall

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Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 86.9 \cdot kip$ Factored Dead Load of Shelter

$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}}$ $CG = 7.5\,ft$ Center of gravity from bottom of shelter

Tie Down Bracket $Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 158.1 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}}$ $F_{turn} = 37 \cdot kip$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 291.1 \cdot kip \cdot ft$ Overtruning Moment

$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2}$ $M_{OT.Resist} = 332.3 \cdot kip \cdot ft$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -41.2 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}}$ $F_{OT.trans} = -4.8 \cdot kip$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans} DL$ $F_{net.slide.trans} = 19.6 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 9.8 \cdot kip$

Maximum Shear Force Per Tie-Down, θ_{min}

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 9.8 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$

$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right)$ $\phi P_{pl.FlrTie} = 26.928 \cdot kip$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.364$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.853$ < 1.0

$$\max\left(\frac{T_{tie.max}}{\phi P_{pl.FlrTie}}\right) + \frac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 1.218$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 1.218$ < 1.2

$chk_{8.Flr.Tie}$ = "NG"

If Tie Down Plates are insufficient, the outer most wall shackles may be utilized for Tie Down to the Truck.

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9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

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Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer: Modular Connections, LLC

b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 6 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 2.7 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 22.12 \quad U_{R3.25} = 0.045$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2)\cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 6 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 2.7 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 27.12 \quad U_{R4.75} = 0.037$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (29.292 \quad 31.292 \quad 33.292 \quad 35.292)\,ft$ Length of Floor

$wgt_{flr}^T = (37.559 \quad 37.469 \quad 37.39 \quad 37.32 \quad NaN \quad NaN \quad NaN \quad NaN \quad NaN)\cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 3.82\cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 4.18\cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.719 \\ 0.68 \\ 16.095 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 18.494 \quad U_F = 0.054$

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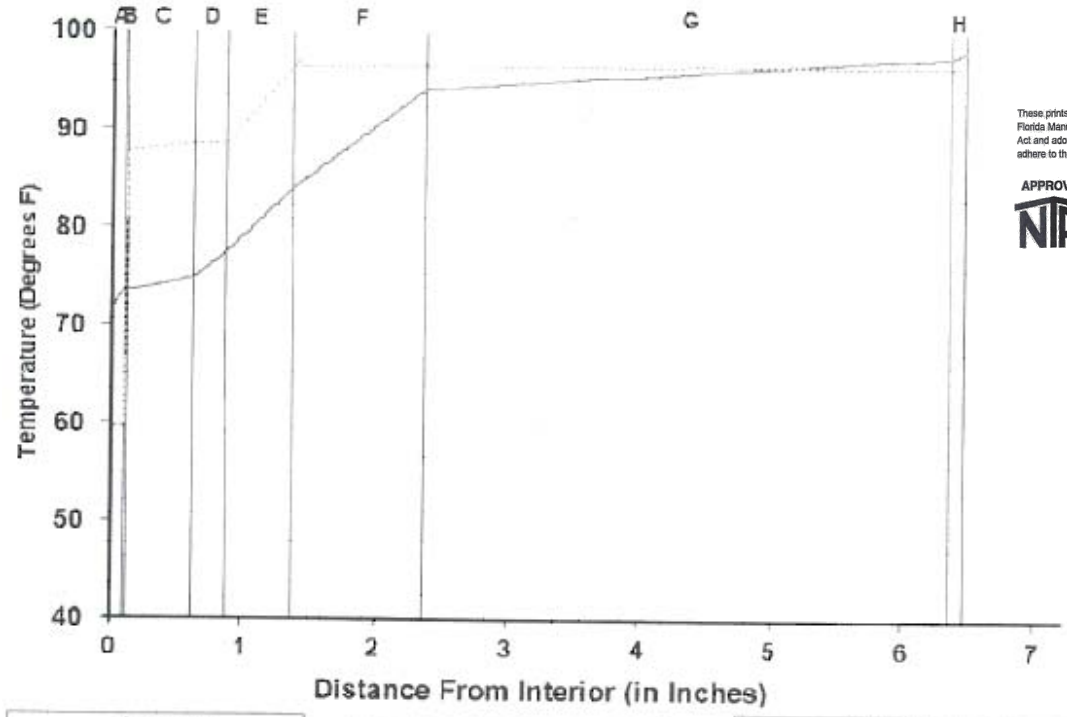
Dewpoint Analysis - Dow Chemical

Sam Cain



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Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

May 17 2006 12:16:28 PM

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10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

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$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 65.753 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 935.2 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 1987 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$ Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.0101$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.242 \cdot in^2$ Req'd Area of Steel per foot Width

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Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 351 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if\left(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG"\right)$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 7.516\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 15.032\,ft$

Maximum allowable opening width

$w_{d.max} := \max\left(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4}\right)$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

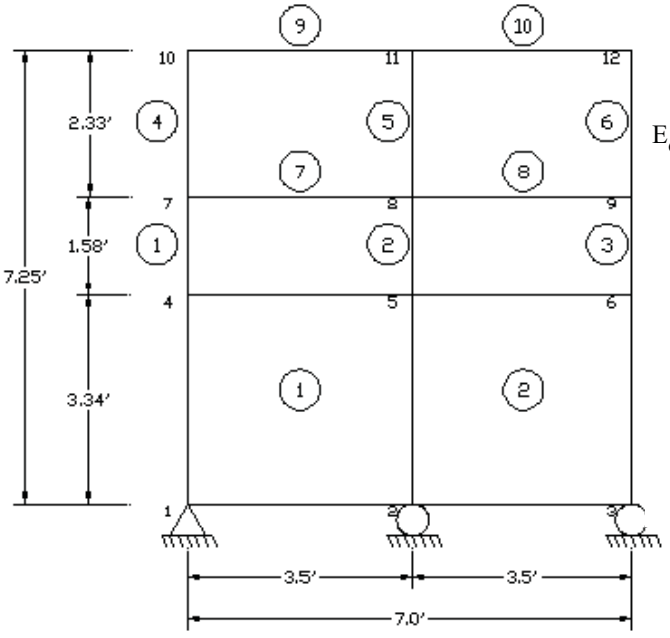
$chk_{opn.width} := if\left(w_d \geq w_{d.max}, "OK", "NG"\right)$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.488 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.488 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.488 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max\left(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2}\right)$ In-Plane Force at Endwall



$V_u = 4.488 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in^2)	I (in^4)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.488 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.465 \cdot kip \cdot ft$

Shear

$\phi Vc_{6d.opn} := \phi_V \cdot 0.85 \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot \left(d_7 - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00114$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.144 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$ Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

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APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPENDIX "B"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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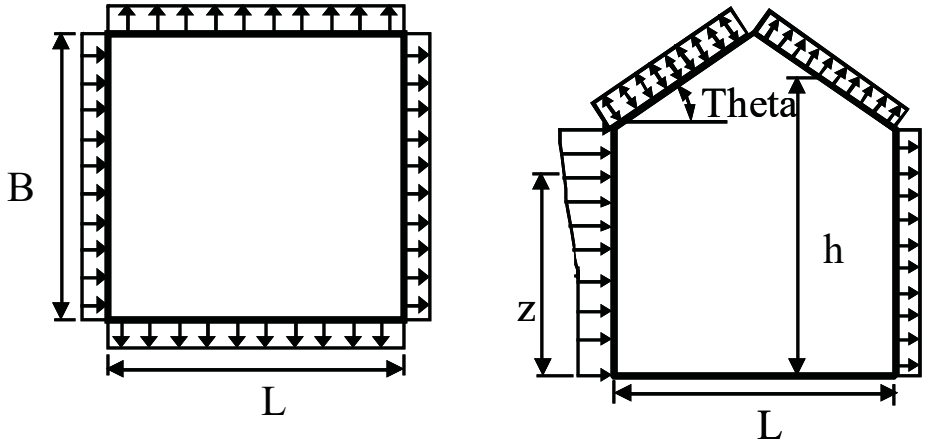
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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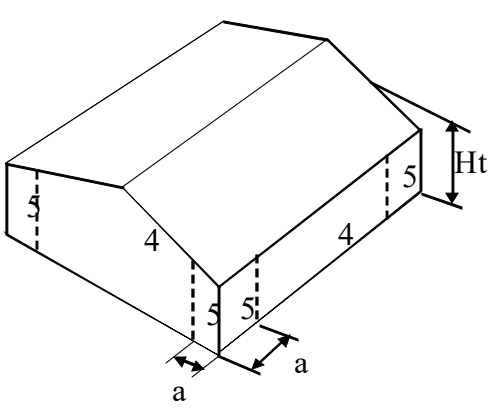
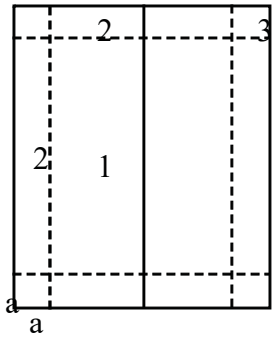
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTION, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Occupancy:	S2
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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPENDIX "C"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

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MODULAR CONNECTIONS, LLC

Cover =

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

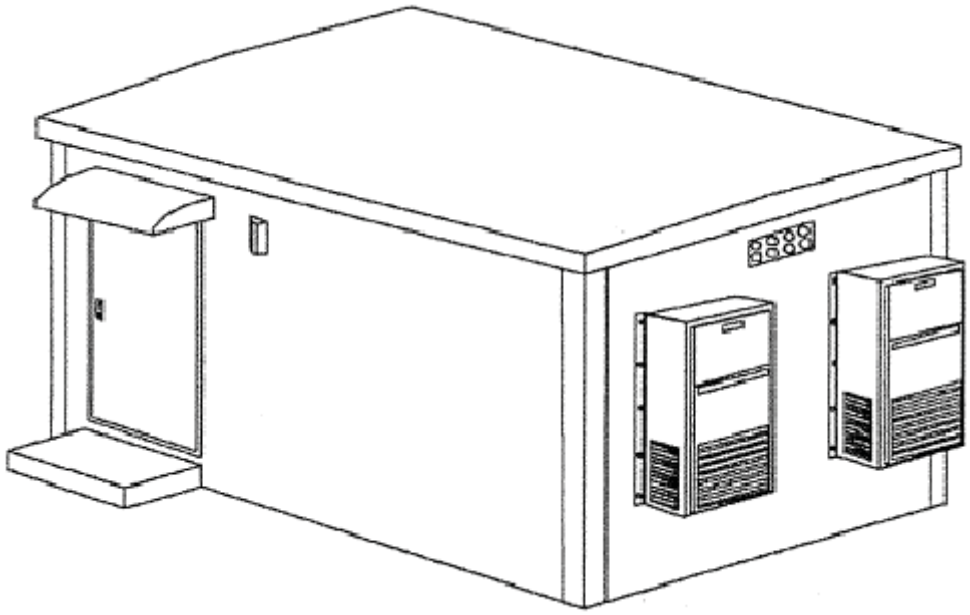
" IBC 2021 (ASCE 7-16) "

" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #2"

" REVISION - 0: DATE MARCH 11, 2024 "

""



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" copied or traced in whole or part without our written consent. "

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" FLOOR DESIGN ---------- S4.1 - S4.31 "
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" FIRE RATING & THERMAL EFFICIENCY---------- S9.1 - S9.3 "
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""
""
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" APPENDIX B - AWINING / HOOD ATTACHMENT DESIGN "
" APPENDIX C - DEVELOPMENT OF REINFORCEMENT "

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Manufacturer:	Modular Connections, LLC

I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.

2024.05.29 15:32:47-04'00'

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1090 Industrial Blvd.
Bessemer, AL 35022
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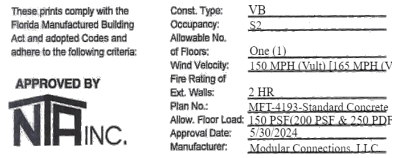

PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 37.4 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 56.5 psf"
"Roof Live Load ---------- 80 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 150 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 35.8 psf"
"Uplift Wind Pressure, MWFRS ---------- 53.5 psf"
"Zone 4 Wind Pressure, C & C ---------- 53.3 psf"
"Zone 5 Wind Pressure, C & C ---------- 65.8 psf"
"Zone 1 Wind Pressure, C & C ---------- 78.2 psf"
"Zone 2 Wind Pressure, C & C ---------- 103.2 psf"
"Zone 3 Wind Pressure, C & C ---------- 140.7 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 32 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 39.5 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.63"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.16"

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Manufacturer:	Modular Connections, LLC

1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$)
Welding electrodes : E70XX
Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"
$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"
$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"
$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 80 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"
$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot psf \qquad q_h = 41.616 \cdot psf \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 20.81 & 35.79 \\ -25.18 & -10.2 \\ -32.25 & -17.27 \\ -53.48 & -38.49 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 35.79 & 20.81 \\ -10.2 & -25.18 \\ -17.27 & -32.25 \\ -38.49 & -53.48 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 35.79 \\ 25.18 \\ 32.25 \\ -53.48 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 45.99 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 41.616 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -65.8 & -50.8 \\ 5 & 20 \\ -78.2 & -63.3 \\ 5 & 20 \\ -103.2 & -88.2 \\ 5 & 20 \\ -140.7 & -125.7 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -50.8 & -65.8 \\ 20 & 5 \\ -63.3 & -78.2 \\ 20 & 5 \\ -88.2 & -103.2 \\ 20 & 5 \\ -125.7 & -140.7 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 53.3 \\ 53.3 \\ 65.8 \\ 78.2 \\ 103.2 \\ 140.7 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 65.8 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 78.2 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 103.2 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 140.7 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 107.369 \cdot psf$$

Max Roof Uplift Pressure (average)

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Const. Type: VB
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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 0.8 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1.18 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 0.944$ $S_{MS.ref} =$ "Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.629$ $S_{DS.ref} =$ "5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} =$ "Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} =$ "5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"

$$H_E = \begin{pmatrix} 9.563 \\ 10.563 \\ 11.229 \end{pmatrix} \text{ft} \qquad H_{E.ref} = \text{"Exterior Height of Building"}$$

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right] \qquad Ta = 0.104 \quad Ta_{ref} = \text{"Approx Per., Eqn (12.8-7)"}$$

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_0 = 0.267$ $T_s := \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_s = 1.333$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.1573$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E}\right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E}\right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1}\right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0277$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0\right)\right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max}\right), C_{s.min}\right)$ $C_s = 0.157$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 5 \cdot in$ Thickness @ ridge $t_{r.r} = 6.5 \cdot in$ $t_{r.a} = 5.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.1404 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 56.54 \cdot psf$ $DL_{r.mn.ref} :=$ "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 63.54 \cdot psf$ $DL_{r.mx.ref} :=$ "Max. Roof DL"

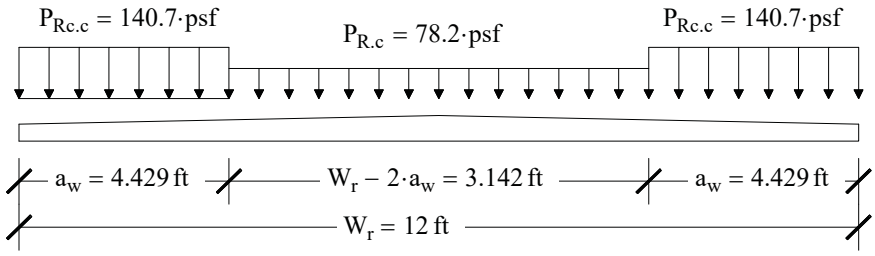
$w_{llr} = 80 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 140.7 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 78.2 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.429\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -24.247 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -112.255 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 7.998 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -7.998 \cdot psf$ Minimum Upward Roof Seismic Load

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$$LRFD = \begin{pmatrix} \text{"1a"} & \text{"1.4(D+F)"} \\ \text{"2a"} & \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"3a"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"3b+"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"3b-"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"4a+"} & \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"4a-"} & \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"5a+"} & \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"5a-"} & \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"6a+"} & \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"6a-"} & \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"7a+"} & \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"7a-"} & \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Wu_{S2.0} = \begin{pmatrix} 88.96 \\ 116.25 \\ 204.25 \\ 204.25 \\ 148.12 \\ 116.25 \\ 60.12 \\ 90.6 \\ 74.6 \\ 50.89 \\ -61.37 \\ 58.89 \\ 42.89 \end{pmatrix} \cdot psf$$

Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref} = \text{"Max Span of Roof Panel"}$

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 204.25 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -61.367 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

	$Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$	$Vu_{ref} = \text{"Ultimate Shear"}$
Roof Live	$Mu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}$	$Vu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}$
Load	$Mu_{2.1.mx} = 37.071 \cdot kip \cdot in$	$Vu_{2.1.mx} = 1.123 \cdot kip$
Option 1	$Mu_{2.1.mn} := \min\left(0kip \cdot in, \lvert w_{u.mn} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$	$Vu_{2.1.mn} := \min\left(0kip, \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}\right)$
	$Mu_{2.1.mn} = 0 \cdot kip \cdot in$	$Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$

$B_{2.1} = \text{"#4"}$ $B_{2.1.ref} = \text{"Transverse Rebar Size"}$ $S_{2.1} = 12 \cdot in$ $S_{2.1.ref} = \text{"Transverse Rebar Spacing"}$

$B_{2.2} = \text{"#4"}$ $B_{2.2.ref} = \text{"Longitudinal Rebar Size"}$ $S_{2.2} = 12 \cdot in$ $S_{2.2.ref} = \text{"Longitudinal Rebar Spacing"}$

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref} = \text{"Clear Cover, From Bot. w/ Keyway"}$

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref} = \text{"Clear Cover, From Bot. w/o Keyway"}$

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref} = \text{"Roof Width Bar Diam."}$ $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref} = \text{"Roof Width Bar Area."}$

$As_{2.1} = 0.2 \cdot in^2$ $As_{2.1.ref} = \text{"Roof Width Area Reinf."}$ $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref} = \text{"Roof Length Bar Diam."}$

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref} = \text{"Roof Length Bar Area."}$ $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref} = \text{"Roof Length Area Reinf."}$

$A_{s.mim.2} = 0.1404 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 4 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.2 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.0042$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.261 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 46.4 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.317 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.03487$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 41.8 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 37.1 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 2.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 3.523 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 13.685 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 3.523 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 2.642 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 1.123 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{F.2.3} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.5} = 0.261 \cdot in$ a_{ref} = "Depth of Comp. Block, Eqn (22.2.2.4.1)"

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $M_{n.2.3} = 19.4 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $c_{2.3} = 0.317 \cdot in$ c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\varepsilon_{2.3} = 0.01357$ ε_{ref} = "Tensile Strain, Extreme Layer"

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ ϕ_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\phi M_{n.2.3} = 17.5 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $Mu_{n.ref}$ = "Ultimate Moment, -ve"

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.5}$ = "OK"

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{V.2.4} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\lambda_{s.2.4} = 1$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.a} = 2.031 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $Vc_{2.4} = 2.031 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\phi Vc_{2.4} = 1.523 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Vu_{2.1.mn} = 0 \cdot kip$ Vu_{ref} = "Ultimate Shear"

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.6}$ = "OK"

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Manufacturer: Modular Connections, LLC

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 190.109 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 2.875 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 28.28 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 26.053 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 11.533 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 95.055 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -6.654 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 47.527 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 47.53 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 190.1 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 78.3 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.03681 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.202 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.16519 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.441\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.083\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.524\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 251.9$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 80 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 80 \cdot psf$

$P_{W.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_0}$ = " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_1}$ = " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 65.8 \cdot psf$ $GC_{p.cc.max.ref_2}$ = "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 53.27 \\ 65.75 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 80 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.1 \\ 1.19 \\ 1.72 \\ 1.72 \\ 2.04 \\ 1.19 \\ 1.51 \\ 1.03 \\ 0.94 \\ 0.62 \\ 1.27 \\ 0.67 \\ 0.58 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 26.6 \\ 26.6 \\ 53.3 \\ 26.6 \\ 7.1 \\ -7.1 \\ 53.3 \\ 53.3 \\ 7.1 \\ -7.1 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 2.04 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.54 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.58 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.07 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.47 \cdot kip$

$P_{u.3b.h1}^T = (1.105 \quad 1.187 \quad 1.715 \quad 1.715 \quad 2.037 \quad 1.187 \quad 1.509 \quad 1.033 \quad 0.937 \quad 0.624 \quad 1.268 \quad 0.672 \quad 0.576) \cdot kip$

$M_{u.3b.h1}^T = (0 \quad 0 \quad 0 \quad 0.27 \quad 0.27 \quad 0.539 \quad 0.27 \quad 0.072 \quad -0.072 \quad 0.539 \quad 0.539 \quad 0.072 \quad -0.072) \cdot kip \cdot ft$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.12 \;\; 0.12 \;\; 0.24 \;\; 0.12 \;\; 0.032 \;\; -0.032 \;\; 0.24 \;\; 0.24 \;\; 0.032 \;\; -0.032)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10ft$ & Roof Live Load $w_{llr} = 80\cdot psf$

$Pu_{3b.h2}^{T} = (1.17 \;\; 1.24 \;\; 1.77 \;\; 1.77 \;\; 2.09 \;\; 1.24 \;\; 1.56 \;\; 1.09 \;\; 0.99 \;\; 0.66 \;\; 1.3 \;\; 0.71 \;\; 0.61)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 7.1 \;\; -7.1 \;\; 53.3 \;\; 53.3 \;\; 7.1 \;\; -7.1)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2}) \qquad P_{u.3b.h2.mx} = 2.09\cdot kip \qquad M_{u.3b.h2.mx} := \max(M_{u.3b.h2}) \qquad M_{u.3b.h2.mx} = 0.67\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2}) \qquad P_{u.3b.h2.mn} = 0.61\cdot kip \qquad M_{u.3b.h2.mn} := \min(M_{u.3b.h2}) \qquad M_{u.3b.h2.mn} = -0.09\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|) \qquad V_{u.3b.h2.mx} = 0.52\cdot kip$

$P_{u.3b.h2}^{T} = (1.168 \;\; 1.242 \;\; 1.77 \;\; 1.77 \;\; 2.092 \;\; 1.242 \;\; 1.564 \;\; 1.088 \;\; 0.992 \;\; 0.659 \;\; 1.304 \;\; 0.707 \;\; 0.611)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.333 \;\; 0.333 \;\; 0.666 \;\; 0.333 \;\; 0.089 \;\; -0.089 \;\; 0.666 \;\; 0.666 \;\; 0.089 \;\; -0.089)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.133 \;\; 0.133 \;\; 0.266 \;\; 0.133 \;\; 0.036 \;\; -0.036 \;\; 0.266 \;\; 0.266 \;\; 0.036 \;\; -0.036)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667ft$ & Roof Live Load $w_{llr} = 80\cdot psf$

$Pu_{3b.h3}^{T} = (1.21 \;\; 1.28 \;\; 1.81 \;\; 1.81 \;\; 2.13 \;\; 1.28 \;\; 1.6 \;\; 1.12 \;\; 1.03 \;\; 0.68 \;\; 1.33 \;\; 0.73 \;\; 0.63)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 7.1 \;\; -7.1 \;\; 53.3 \;\; 53.3 \;\; 7.1 \;\; -7.1)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3}) \qquad P_{u.3b.h3.mx} = 2.13\cdot kip \qquad M_{u.3b.h3.mx} := \max(M_{u.3b.h3}) \qquad M_{u.3b.h3.mx} = 0.76\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3}) \qquad P_{u.3b.h3.mn} = 0.63\cdot kip \qquad M_{u.3b.h3.mn} := \min(M_{u.3b.h3}) \qquad M_{u.3b.h3.mn} = -0.1\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|) \qquad V_{u.3b.h3.mx} = 0.56\cdot kip$

$P_{u.3b.h3}^{T} = (1.211 \;\; 1.278 \;\; 1.806 \;\; 1.806 \;\; 2.128 \;\; 1.278 \;\; 1.6 \;\; 1.124 \;\; 1.028 \;\; 0.683 \;\; 1.327 \;\; 0.731 \;\; 0.635)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.379 \;\; 0.379 \;\; 0.758 \;\; 0.379 \;\; 0.101 \;\; -0.101 \;\; 0.758 \;\; 0.758 \;\; 0.101 \;\; -0.101)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.142 \;\; 0.142 \;\; 0.284 \;\; 0.142 \;\; 0.038 \;\; -0.038 \;\; 0.284 \;\; 0.284 \;\; 0.038 \;\; -0.038)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\qquad S_{3.0}$ = "N/A" $\cdot in \qquad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \qquad d_{3b} := \frac{t_w}{2} - 0.25in \qquad d_{3b} = 1.75\cdot in$

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.555 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.076$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.929$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

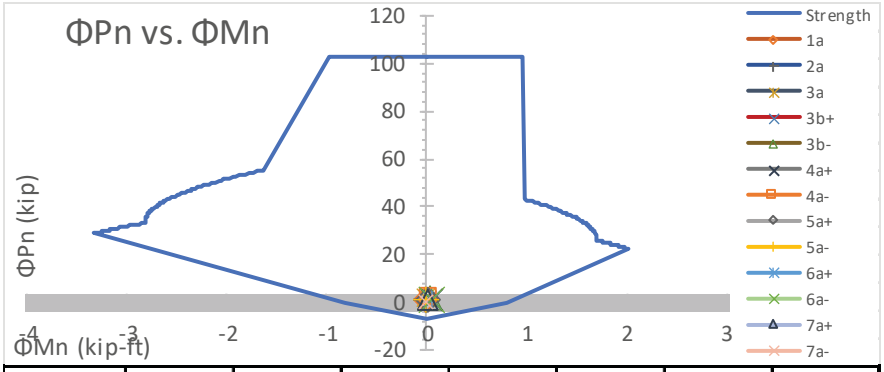
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.098$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.91$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.116$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.896$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.896$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.2%	1.2%	1.8%	2.94%	2.9%	6.1%	3.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.1%	1.0%	6.3%	6.0%	0.8%	0.8%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref} = "Min.\ Space\ to\ Adjacent\ Opening"$ $b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref} = "Strip\ Width\ Btwn\ Openings"$

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1} = "\#6"$ $B_{3.1.ref} = "Wall\ Vert.\ Perim.\ Reinf."$ $B_{3.2} = "\#4"$ $B_{3.2.ref} = "Wall\ Top.\ Perim.\ Reinf."$

$N_{B.3.1} = 1$ $N_{B.3.1.ref} = "\#\ Wall\ Vert.\ Perim.\ Bars"$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = "\#\ Wall\ Top\ Perim.\ Bars"$

$D_{3c.1} = 0.75 \cdot in$ $D_{ref} = "Bar\ Diam."$ $D_{3c.2} = 0.5 \cdot in$ $D_{ref} = "Bar\ Diam."$

$A_{3c.1} = 0.44 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.3} = "\#6"$ $B_{3.3.ref} = "EW\ Bot\ Perim.\ Reinf."$ $B_{3.4} = "\#4"$ $B_{3.4.ref} = "SW\ Bot\ Perim.\ Reinf."$

$N_{B.3.3} = 1$ $N_{B.3.3.ref} = "\#\ EW\ Bot\ Perim.\ Bars"$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = "\#\ SW\ Bot\ Perim.\ Bars"$

$D_{3c.3} = 0.75 \cdot in$ $D_{ref} = "Bar\ Diam."$ $D_{3c.4} = 0.5 \cdot in$ $D_{ref} = "Bar\ Diam."$

$A_{3c.3} = 0.44 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$ $A_{3c.4} = 0.4 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.5} = "\#5"$ $B_{3.5.ref} = "Wall\ Opening\ Reinf."$ $D_{3c.5} = 0.625 \cdot in$ $D_{ref} = "Bar\ Diam."$

$N_{B.3.5} = 2$ $N_{B.3.5.ref} = "\#\ Wall\ Opening\ Bars"$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 80 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}{}^T = (0\ \ 0\ \ 0\ \ 106.5\ \ 106.5\ \ 213.1\ \ 106.5\ \ 28.5\ \ -28.5\ \ 213.1\ \ 213.1\ \ 28.5\ \ -28.5) \cdot plf$

$P_{u.3c.h1d1}{}^T = (4.42\ \ 4.75\ \ 6.86\ \ 6.86\ \ 8.15\ \ 4.75\ \ 6.04\ \ 4.13\ \ 3.75\ \ 2.5\ \ 5.07\ \ 2.69\ \ 2.3) \cdot kip$

$M_{u.3c.h1d1}{}^T = (0\ \ 0\ \ 0\ \ 0.65\ \ 0.65\ \ 1.31\ \ 0.65\ \ 0.17\ \ -0.17\ \ 1.31\ \ 1.31\ \ 0.17\ \ -0.17) \cdot kip \cdot ft$

$V_{u.3c.h1d1}{}^T = (0\ \ 0\ \ 0\ \ 0.37\ \ 0.37\ \ 0.75\ \ 0.37\ \ 0.1\ \ -0.1\ \ 0.75\ \ 0.75\ \ 0.1\ \ -0.1) \cdot kip$

$P_{u.ref} = "Fact.\ Axial\ Load"$ $M_{u.ref} = "Fact.\ Moment"$ $V_{u.ref} = "Fact.\ Shear\ Load"$

$P_{u.3c.h1d1.mx} = 8.15 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.31 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.46 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.3 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.17 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 80 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$W_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 21.4 \;\; -21.4 \;\; 159.8 \;\; 159.8 \;\; 21.4 \;\; -21.4)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.31 \;\; 3.56 \;\; 5.15 \;\; 5.15 \;\; 6.11 \;\; 3.56 \;\; 4.53 \;\; 3.1 \;\; 2.81 \;\; 1.87 \;\; 3.8 \;\; 2.02 \;\; 1.73)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.13 \;\; -0.13 \;\; 0.98 \;\; 0.98 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.07 \;\; -0.07 \;\; 0.56 \;\; 0.56 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 6.11\cdot kip$ $M_{u.3c.h1d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.09\cdot kip$

$P_{u.3c.h1d2.mn} = 1.73\cdot kip$ $M_{u.3c.h1d2.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 17.8 \;\; -17.8 \;\; 133.2 \;\; 133.2 \;\; 17.8 \;\; -17.8)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.76 \;\; 2.97 \;\; 4.29 \;\; 4.29 \;\; 5.09 \;\; 2.97 \;\; 3.77 \;\; 2.58 \;\; 2.34 \;\; 1.56 \;\; 3.17 \;\; 1.68 \;\; 1.44)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.11 \;\; -0.11 \;\; 0.82 \;\; 0.82 \;\; 0.11 \;\; -0.11)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.06 \;\; -0.06 \;\; 0.47 \;\; 0.47 \;\; 0.06 \;\; -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 5.09\cdot kip$ $M_{u.3c.h1d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 0.91\cdot kip$

$P_{u.3c.h1d3.mn} = 1.44\cdot kip$ $M_{u.3c.h1d3.mn} = -0.11\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 28.5 \;\; -28.5 \;\; 213.1 \;\; 213.1 \;\; 28.5 \;\; -28.5)\cdot plf$

$P_{u.3c.h2d1}^{T} = (4.67 \;\; 4.97 \;\; 7.08 \;\; 7.08 \;\; 8.37 \;\; 4.97 \;\; 6.25 \;\; 4.35 \;\; 3.97 \;\; 2.64 \;\; 5.21 \;\; 2.83 \;\; 2.45)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.17 \;\; -0.17 \;\; 1.31 \;\; 1.31 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.1 \;\; -0.1 \;\; 0.75 \;\; 0.75 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 8.37\cdot kip$ $M_{u.3c.h2d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.46\cdot kip$

$P_{u.3c.h2d1.mn} = 2.45\cdot kip$ $M_{u.3c.h2d1.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 21.4 \;\; -21.4 \;\; 159.8 \;\; 159.8 \;\; 21.4 \;\; -21.4)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.51 \;\; 3.72 \;\; 5.31 \;\; 5.31 \;\; 6.27 \;\; 3.72 \;\; 4.69 \;\; 3.26 \;\; 2.97 \;\; 1.98 \;\; 3.91 \;\; 2.12 \;\; 1.83)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.13 \;\; -0.13 \;\; 0.98 \;\; 0.98 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.07 \;\; -0.07 \;\; 0.56 \;\; 0.56 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 6.27\cdot kip$ $M_{u.3c.h2d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.09\cdot kip$

$P_{u.3c.h2d2.mn} = 1.83\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 17.8 \;\; -17.8 \;\; 133.2 \;\; 133.2 \;\; 17.8 \;\; -17.8)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (2.92 \;\; 3.1 \;\; 4.42 \;\; 4.42 \;\; 5.23 \;\; 3.1 \;\; 3.91 \;\; 2.72 \;\; 2.48 \;\; 1.65 \;\; 3.26 \;\; 1.77 \;\; 1.53)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.11 \;\; -0.11 \;\; 0.82 \;\; 0.82 \;\; 0.11 \;\; -0.11)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.06 \;\; -0.06 \;\; 0.47 \;\; 0.47 \;\; 0.06 \;\; -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 5.23\cdot kip$ $M_{u.3c.h2d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 0.91\cdot kip$

$P_{u.3c.h2d3.mn} = 1.53\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.11\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 28.5 \;\; -28.5 \;\; 213.1 \;\; 213.1 \;\; 28.5 \;\; -28.5)\cdot plf$

$P_{u.3c.h3d1}^{T} = (4.84 \;\; 5.11 \;\; 7.22 \;\; 7.22 \;\; 8.51 \;\; 5.11 \;\; 6.4 \;\; 4.5 \;\; 4.11 \;\; 2.73 \;\; 5.31 \;\; 2.92 \;\; 2.54)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.17 \;\; -0.17 \;\; 1.31 \;\; 1.31 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.1 \;\; -0.1 \;\; 0.75 \;\; 0.75 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 8.51\cdot kip$ $M_{u.3c.h3d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.46\cdot kip$

$P_{u.3c.h3d1.mn} = 2.54\cdot kip$ $M_{u.3c.h3d1.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 21.4 \;\; -21.4 \;\; 159.8 \;\; 159.8 \;\; 21.4 \;\; -21.4)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.63 \;\; 3.83 \;\; 5.42 \;\; 5.42 \;\; 6.38 \;\; 3.83 \;\; 4.8 \;\; 3.37 \;\; 3.08 \;\; 2.05 \;\; 3.98 \;\; 2.19 \;\; 1.9)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.13 \;\; -0.13 \;\; 0.98 \;\; 0.98 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.07 \;\; -0.07 \;\; 0.56 \;\; 0.56 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 6.38\cdot kip$ $M_{u.3c.h3d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.09\cdot kip$

$P_{u.3c.h3d2.mn} = 1.9\cdot kip$ $M_{u.3c.h3d2.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 17.8 \;\; -17.8 \;\; 133.2 \;\; 133.2 \;\; 17.8 \;\; -17.8)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.11 \;\; -0.11 \;\; 0.82 \;\; 0.82 \;\; 0.11 \;\; -0.11)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.06 \;\; -0.06 \;\; 0.47 \;\; 0.47 \;\; 0.06 \;\; -0.06)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 5.32\cdot kip$ $M_{u.3c.h3d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 0.91\cdot kip$

$P_{u.3c.h3d3.mn} = 1.59\cdot kip$ $M_{u.3c.h3d3.mn} = -0.11\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - (0.75in + 0.5D_{3c.5})\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.458\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.093\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 0.911\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{(k_{wall}\cdot H_{wall})^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

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$\delta_{3c.h1d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.394$ $\Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d1} = 0.717$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

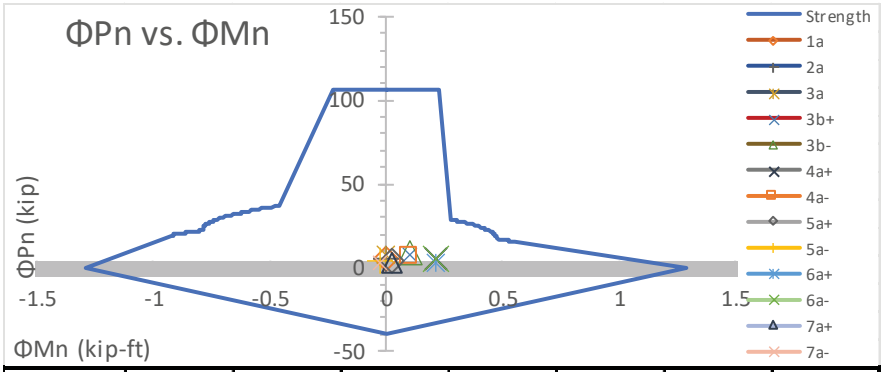
$\delta_{3c.h2d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d1} = 1.559$ $\Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1}$ $\Omega_{\delta.3c.h2d1} = 0.642$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h3d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d1} = 2.649$ $\Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1}$ $\Omega_{\delta.3c.h3d1} = 0.378$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3c.d1} := \min(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1})$ $\Omega_{\delta.3c.d1} = 0.378$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.5%	4.8%	6.8%	11.41%	12.2%	20.5%	10.9%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.2%	3.9%	18.5%	20.7%	2.7%	2.4%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h1d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.394$ $\Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d2} = 0.717$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h2d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d2} = 1.368$ $\Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1}$ $\Omega_{\delta.3c.h2d2} = 0.731$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h3d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d2} = 1.876$ $\Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1}$ $\Omega_{\delta.3c.h3d2} = 0.533$

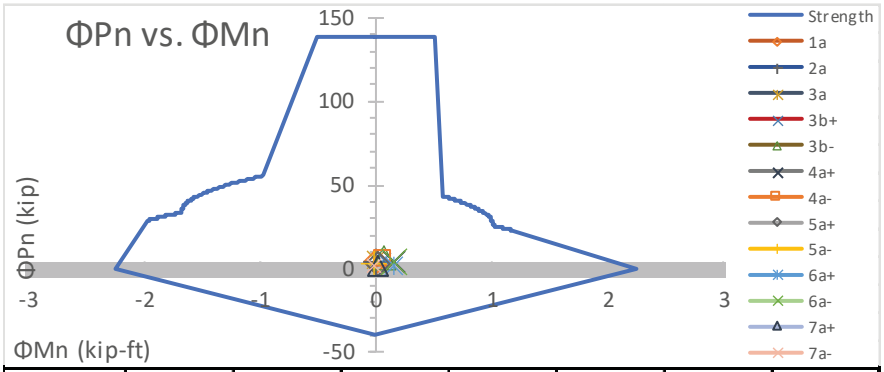
Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\Omega_{\delta.3c.d2} := \min(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2})$ $\Omega_{\delta.3c.d2} = 0.533$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.6%	2.8%	3.9%	4.01%	4.6%	7.7%	4.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.4%	2.2%	7.4%	7.8%	1.6%	1.4%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.215 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.823$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

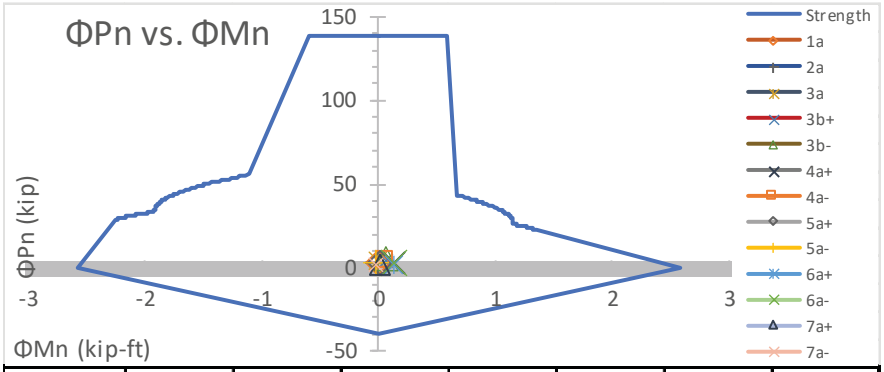
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.289 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.776$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.637 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.611$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.611$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.2%	2.3%	3.3%	3.26%	3.8%	5.5%	2.9%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.0%	1.9%	5.4%	5.6%	1.3%	1.1%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

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d. Wall - Header Check (Above Opening) $w_{llr} = 80 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in$ Min Header Height for Opening Widths of $w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$Wu_{3d.0}^{T} = (745.3 \quad 878.8 \quad 1406.8 \quad 1406.8 \quad 1728.9 \quad 878.8 \quad 1200.9 \quad 724.9 \quad 628.9 \quad 423.3 \quad 1067.5 \quad 471.3 \quad 375.3) \cdot plf$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1728.9 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in$ $A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for for:
$w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min(Vc_{3d.0.a}, Vc_{3d.0.b})$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot (w_{open_0})^2}{8}$ $Mu_{3d.0} = 7.78 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max(|wu_{3d.0mx}|, |wu_{3d.0mn}|) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 5.19 \cdot kip$

$chk_{3d.1} := if(\phi Mn_{3d.0} \geq Mu_{3d.0}, "OK", "NG")$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot [f_y \cdot (0.5 \cdot h_{3d_0})]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := if(chk_{3d.2} = "OK", "OK", if(chk_{3d.2} = "Stirrups Req'd", if(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, "NG", "OK"), "OK"))$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (644.8 \;\; 792.7 \;\; 1320.7 \;\; 1320.7 \;\; 1642.8 \;\; 792.7 \;\; 1114.8 \;\; 638.8 \;\; 542.8 \;\; 367.3 \;\; 1011.5 \;\; 415.3 \;\; 319.3) \cdot plf$

$wu_{3d.1mx} := \max(Wu_{3d.1})$ $wu_{3d.1mx} = 1642.8 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min(Wu_{3d.1}, 0plf)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}(a_{3d.1.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots$$
$$+ A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := root(f_{3d.1}(a_{3d.1.g}), a_{3d.1.g})$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F2_{3d.1} := A_{3d.1_1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.1} = 11.7\cdot kip$$

$$F3_{3d.1} := A_{3d.1_2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.1} = 7.2\cdot kip$$

$$F4_{3d.1} := A_{3d.1_3}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.1} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772\cdot kip$$

$$Mn_{3d.1} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.1}\cdot\frac{a_{3d.1}}{2} + F1_{3d.1}\cdot d_{3d.1_0} + F2_{3d.1}\cdot d_{3d.1_1} \ldots + F3_{3d.1}\cdot d_{3d.1_2} + F4_{3d.1}\cdot d_{3d.1_3} \qquad Mn_{3d.1} = 38.5\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}}\cdot d_{3d.1_3} \qquad \varepsilon_{3d.1} = 0.014 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.1} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.1} := \phi_{3d.1}\cdot Mn_{3d.1} \qquad \phi Mn_{3d.1} = 34.64\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375\cdot in$$

$$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right)\cdot t_w} \qquad \rho_{3d.1} = 0.0062$$

$$\rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{3d.1} := \sum\begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.1_0}\right)}\right] \qquad \rho'_{3d.1} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.1} = 0.876$$

$$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$$

$$Vc_{3d.1.a} := 8\cdot\lambda_{s.3d.1}\cdot\lambda\cdot\rho_{3d.1}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.a} = 5.48\cdot kip$$

$$Vc_{3d.1.b} = 21.31\cdot kip$$

$$\phi Vc_{3d.1} := \phi_V\cdot\min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right) \qquad \phi Vc_{3d.1} = 4.1\cdot kip \qquad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$$

$$Mu_{3d.1} := \frac{wu_{3d.1mx}\cdot\left(w_{open_1}\right)^2}{8} \qquad Mu_{3d.1} = 3.29\cdot kip\cdot ft$$

$$Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right)\cdot w_{open_1}}{2} \qquad Vu_{3d.1} = 3.29\cdot kip$$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h_{3d_1}\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (623.7 \quad 774.6 \quad 1302.6 \quad 1302.6 \quad 1624.7 \quad 774.6 \quad 1096.7 \quad 620.7 \quad 524.7 \quad 355.5 \quad 999.7 \quad 403.5 \quad 307.5) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1624.7 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_2} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_2} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 3.29 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 2.44 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} :=$ "OK" if $\phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2}$; "Stirrups Req'd" otherwise $\quad chk_{3d.10} = \text{"Min. Stirrups Req'd"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 2.98 \cdot psf$		$M_{E.4a} = 0.001 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.002 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.234 \\ 0.234 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.08 \\ 0.08 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.193 \\ 0.193 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.063 \\ 0.063 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.151 \\ 0.151 \\ 0.015 \\ 0.015 \\ 0.017 \\ 0.017 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PSF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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1090 Industrial Blvd.
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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 9.19 \cdot plf$		$M_{E.4b} = 0.1 \cdot kip \cdot ft$	$V_{E.4b} = 0.043 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.5 \\ 6.5 \\ 0.58 \\ 0.58 \\ 0.68 \\ 0.68 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.79 \\ 2.79 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.41 \\ 5.41 \\ 0.58 \\ 0.58 \\ 0.68 \\ 0.68 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.32 \\ 2.32 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.32 \\ 4.32 \\ 0.58 \\ 0.58 \\ 0.68 \\ 0.68 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.85 \\ 1.85 \\ 0.25 \\ 0.25 \\ 0.29 \\ 0.29 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Const. Type: VB
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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (12 \quad 13 \quad 14 \quad 15)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (29.29 \quad 31.29 \quad 33.29 \quad 35.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (6.86 \quad 6.96 \quad 7.04 \quad 7.12) \cdot psf$ Floor Transverse Ribs

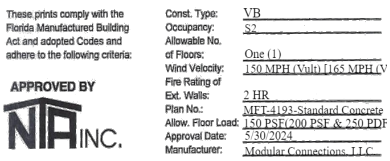
$wgt_{f.l.rib}^T = (3.63 \quad 3.63 \quad 3.64 \quad 3.64) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (3.27 \quad 3.06 \quad 2.88 \quad 2.72) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (4.13 \quad 4.15 \quad 4.16 \quad 4.17) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (37.6 \quad 37.5 \quad 37.4 \quad 37.3) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (41.6 \quad 41.5 \quad 41.4 \quad 41.3) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"



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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

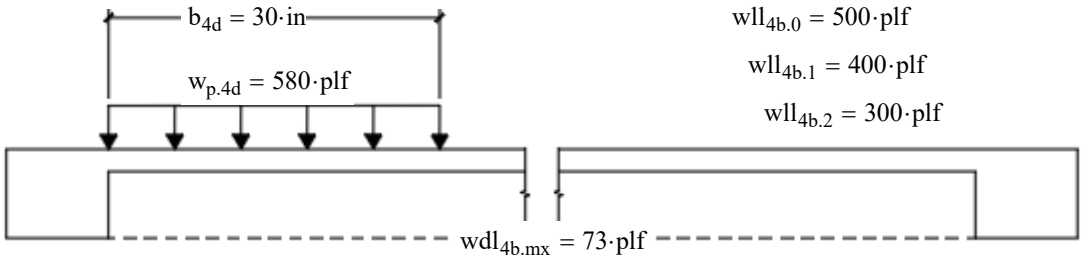
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

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APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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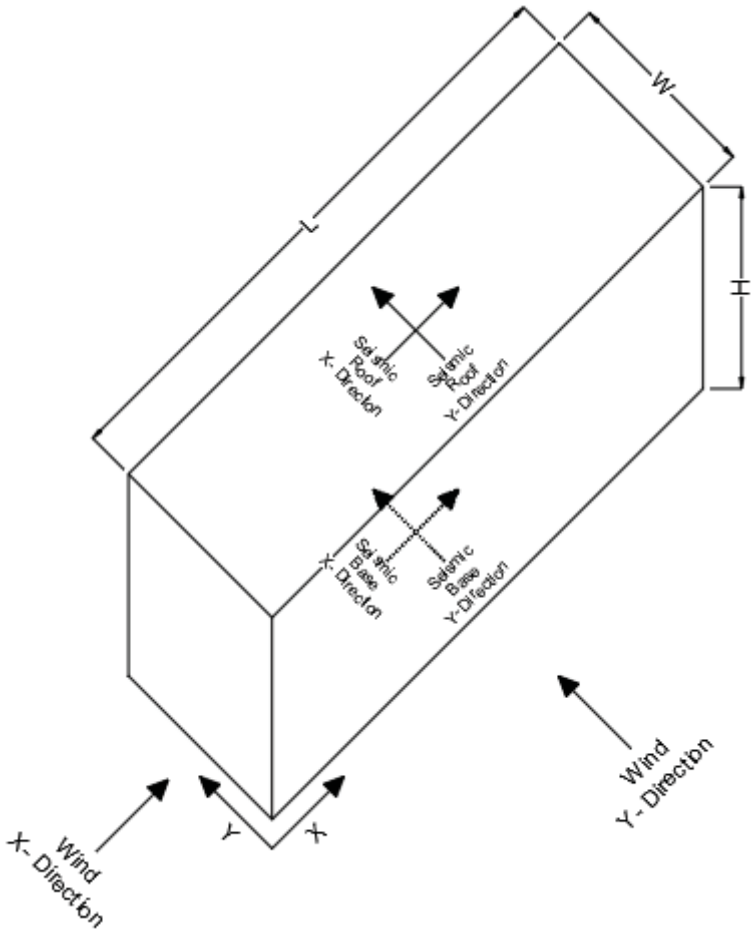
5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx} \cdot (L_r) \cdot (W_r)$

$Wt_{rf.mn} := DL_{r.mn} \cdot (L_r) \cdot (W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25 \cdot w_{llr}, 0psf)$

$w_{llr.eq} = 20 \cdot psf$

$Wt_{25ll} := w_{llr.eq} \cdot [(L_r) \cdot (W_r)]$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{rf.mx}^{T} = (23.1 \quad 24.7 \quad 26.2 \quad 27.7) \cdot kip$

Max Roof Weight, $w_{llr} = 80 \cdot psf$

$Wt_{25ll}^{T} = (7.3 \quad 7.8 \quad 8.2 \quad 8.7) \cdot kip$

25% Roof LLr 0, $w_{llr} = 80 \cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (30 \quad 32 \quad 34 \quad 36) \cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (20.6 \quad 21.9 \quad 23.3 \quad 24.7) \cdot kip$

Min Roof Weight, $w_{llr} = 80 \cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75 \cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3} \cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128) \cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (114.5 \quad 126.5 \quad 134.5) \cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 111.25 \\ 123.25 \\ 131.25 \end{pmatrix} \cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r} \cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r} \cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r} \cdot el_{avg.rf_2}$

Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h1.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h2.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h0.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h1.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h2.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{EQ.w.h0}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h1}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h2}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

Max Wall Weight, $H_{wall_0} = 9\,ft$

Max Wall Weight, $H_{wall_1} = 10\,ft$

Max Wall Weight, $H_{wall_2} = 10.7\,ft$

Min Wall Weight, $H_{wall_0} = 9\,ft$

Min Wall Weight, $H_{wall_1} = 10\,ft$

Min Wall Weight, $H_{wall_2} = 10.7\,ft$

Seismic Wall Weight, $H_{wall_0} = 9\,ft$

Seismic Wall Weight, $H_{wall_1} = 10\,ft$

Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$el_{top.w} := h_{wl\_fl} + H_{wall}$

$el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$

Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean\left(h_{wl\_fl}, el_{top.w_0}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_1}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_2}\right) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$

$W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$

$W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if\left(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf\right)$

$w_{llf.0.eq} = 62.5 \cdot psf$

$Wt_{25llf0} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.1.eq} := if\left(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf\right)$

$w_{llf.1.eq} = 50 \cdot psf$

$Wt_{25llf1} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.2.eq} := if\left(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf\right)$

$w_{llf.2.eq} = 37.5 \cdot psf$

$Wt_{25llf2} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (13.3 \quad 14.2 \quad 15.1 \quad 16) \cdot kip$

Max Floor Weight

$Wt_{f.mn}^T = (12.1 \quad 12.8 \quad 13.6 \quad 14.4) \cdot kip$

Min Floor Weight

$Wt_{25llf0}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$

Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

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$Wt_{EQ.f.f0}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 45.99\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (12.9 \quad 13.8 \quad 14.6 \quad 15.5)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (14.2 \quad 15.2 \quad 16.2 \quad 17.1)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (15.1 \quad 16.2 \quad 17.2 \quad 18.2)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 5.13\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 5.67\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 6.02\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (6.4 \quad 6.9 \quad 7.3 \quad 7.8)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (7.1 \quad 7.6 \quad 8.1 \quad 8.6)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (7.6 \quad 8.1 \quad 8.6 \quad 9.1)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 2.57\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 2.83\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 3.01\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.157$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{EQ.bh0f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh1f0} := C_s\cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh2f0}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh2f0} := C_s\cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh0f1}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh0f1} := C_s\cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh1f1}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh1f1} := C_s\cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{EQ.bh2f1}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh2f1} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh0f2}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh0f2} := C_s\cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh1f2}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh1f2} := C_s\cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.bh2f2}^{T} = (97 \quad 103 \quad 109 \quad 115)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh2f2} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (15 \quad 16 \quad 17 \quad 18)\cdot kip$

$Wt_{EQ.rh0}^{T} = (47 \quad 50 \quad 53 \quad 56)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 80\cdot psf \quad H_{wall_0} = 9\,ft$

$V_{EQ.rh0} := C_s\cdot Wt_{EQ.rh0}$

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$V_{EQ.rh0}^{T} = (7 \quad 8 \quad 8 \quad 9)\cdot kip$

$Wt_{EQ.rh1}^{T} = (47 \quad 50 \quad 53 \quad 56)\cdot kip$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^{T} = (7 \quad 8 \quad 8 \quad 9)\cdot kip$

$Wt_{EQ.rh2}^{T} = (47 \quad 50 \quad 53 \quad 56)\cdot kip$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^{T} = (7 \quad 8 \quad 8 \quad 9)\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Roof Diap. Wt/Shear $w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

Roof Diap. Wt/Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r}$ $w_{EQh0.y}^{T} = (244 \quad 243 \quad 242 \quad 241)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r}$ $w_{EQh1.y}^{T} = (244 \quad 243 \quad 242 \quad 241)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r}$ $w_{EQh2.y}^{T} = (244 \quad 243 \quad 242 \quad 241)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_2} = 10.67ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r}$ $w_{EQh0.x}^{T} = (618 \quad 655 \quad 692 \quad 729)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r}$ $w_{EQh1.x}^{T} = (618 \quad 655 \quad 692 \quad 729)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r}$ $w_{EQh2.x}^{T} = (618 \quad 655 \quad 692 \quad 729)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_2} = 10.67ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r}$ $w_{W.h0.y}^{T} = (212 \quad 213 \quad 213 \quad 214)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r}$ $w_{W.h1.y}^{T} = (235 \quad 235 \quad 235 \quad 236)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r}$ $w_{W.h2.y}^{T} = (249 \quad 250 \quad 250 \quad 251)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$

$w_{W.h0.x} = 213.8\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$

$w_{W.h1.x} = 236.1\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$

$w_{W.h2.x} = 251\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_2} = 10.7\,ft$

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$M_{u.bh0.y}^{T} = (28.1 \quad 31.8 \quad 35.6 \quad 39.7)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (28.1 \quad 31.8 \quad 35.6 \quad 39.7)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (28.1 \quad 31.8 \quad 35.6 \quad 39.7)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.y}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.bh0.x}^{T} = (11.1 \quad 11.8 \quad 12.5 \quad 13.1)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (11.1 \quad 11.8 \quad 12.5 \quad 13.1)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (11.1 \quad 11.8 \quad 12.5 \quad 13.1)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.x}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (3.7 \quad 3.9 \quad 4.2 \quad 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.b.h0.y}^{T} = (24.4 \quad 27.8 \quad 31.4 \quad 35.2)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (27 \quad 30.7 \quad 34.7 \quad 38.9)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (28.7 \quad 32.7 \quad 36.9 \quad 41.4)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{u.b.h0.y}^{T} = (3.2 \quad 3.4 \quad 3.7 \quad 3.9)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (3.6 \quad 3.8 \quad 4 \quad 4.3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.6)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 3.8\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 4.3\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 4.5\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7ft$

$v_{u.b.h0.x} = 1.3\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.4\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = \text{"\#6"}$ $B_{7.1.ref} = \text{"Roof Perim. Reinf."}$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = \text{"\# Roof Perim. Bars"}$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = \text{"Roof Perim. Reiinf. Bar Diam."}$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = \text{"Roof Perim. Reinf. Area"}$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := \text{"Roof Diaph.Reinf."}$ $\rho_{b.1} := \frac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00089$

$d_{b.1} := W_r - 0.75in - \frac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \frac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 2.01\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \frac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\frac{d_{b.1}}{2} - \frac{a_{b.1}}{2}\right)$ $M_{b.1} = 382.5\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \frac{a_{b.1}}{0.85}$ $c_{b.1} = 2.4\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\frac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.17845$ Strain in Ten. Rebar

$\phi_{b.1} := \text{if}\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 344.3\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\frac{M_{u.bh0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.bh1.y}}{\phi M_{b.1_1}} \quad \frac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 8.2 & 9.2 & 10.3 & 11.5 \\ 8.2 & 9.2 & 10.3 & 11.5 \\ 8.2 & 9.2 & 10.3 & 11.5 \end{pmatrix}\cdot\%$

$chk_{b.1a} := \text{if}\left(\max\left(Unity_{b.1a}\right) < 1, \text{"OK"}, \text{"NG"}\right)$

$chk_{b.1a} = \text{"OK"}$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.1b}^T = \begin{pmatrix} 7.1 & 8.1 & 9.1 & 10.2 \\ 7.8 & 8.9 & 10.1 & 11.3 \\ 8.3 & 9.5 & 10.7 & 12 \end{pmatrix} \cdot \%$$

$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.64 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 2.008 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$d_{b.2}^T = (30.26 \quad 32.26 \quad 34.26 \quad 36.26)\,ft$

$\rho_{b.2}^T = (0.00035 \quad 0.00033 \quad 0.00031 \quad 0.00029)$

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.1}}{2} \right)$$

$M_{b.2a}^T = (977 \quad 1042 \quad 1107 \quad 1172) \cdot kip \cdot ft$ Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 2.4 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$\varepsilon_{t.b.2a}^T = (0.4582 \quad 0.4887 \quad 0.5191 \quad 0.5496)$ Strain in Rebar

$\phi_{b.2} := if\left[min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50 \right] \right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^T = (880 \quad 938 \quad 996 \quad 1054) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.2a}^T = \begin{pmatrix} 1.3 & 1.3 & 1.3 & 1.2 \\ 1.3 & 1.3 & 1.3 & 1.2 \\ 1.3 & 1.3 & 1.3 & 1.2 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.2b}^T = \begin{pmatrix} 0.4 & 0.4 & 0.4 & 0.4 \\ 0.5 & 0.5 & 0.4 & 0.4 \\ 0.5 & 0.5 & 0.5 & 0.4 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 720 \cdot in^2$ Roof Shear Area for $t_{r.e} = 5 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00089$

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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e} \cdot L_{r_0} \\ t_{r.e} \cdot L_{r_1} \\ t_{r.e} \cdot L_{r_2} \\ t_{r.e} \cdot L_{r_3} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 1820 \\ 1940 \\ 2060 \\ 2180 \end{pmatrix} \cdot in^2$$

Roof Shear Area for $t_{r.e} = 5 \cdot in$ & $L_r = \begin{pmatrix} 4368 \\ 4656 \\ 4944 \\ 5232 \end{pmatrix} \cdot \frac{in^2}{ft}$

$\rho_{b.2}^{T} = (0.00035 \quad 0.00033 \quad 0.00031 \quad 0.00029)$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv} \cdot \left(2 \cdot \sqrt{f'_c} + \rho \cdot f_y\right)$ $V_{b.3a.1.mx} = 135 \cdot kip$

Max Allowable for $t_{r.e} = 5 \cdot in$ & $W_r = 12 \cdot ft$

$V_{b.3a.2.mx}^{T} = (282.6 \quad 298.7 \quad 314.8 \quad 330.9) \cdot kip$

Max Allowable Shear for $t_{r.e} = 5 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$V_{b.3b.1} = 41.1 \cdot kip$ Nominal Shear Strength for $t_{r.e} = 5 \cdot in$ & $W_r = 12 \cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

Roof Diaph. Shear Strength for $t_{r.e} = 5 \cdot in$ & $W_r = 12 \cdot ft$ $\phi V_{c.b.3a.1} = 30.8 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$V_{b.3b.2}^{T} = (103.8 \quad 110.6 \quad 117.5 \quad 124.3) \cdot kip$

Nominal Shear Strength for $t_{r.e} = 5 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$\phi V_{c.b.3a.2}^{T} = (77.8 \quad 83 \quad 88.1 \quad 93.2) \cdot kip$

Roof Diaph. Shear Strength for $t_{r.e} = 5 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 12 & 12.8 & 13.5 & 14.2 \\ 12 & 12.8 & 13.5 & 14.2 \\ 12 & 12.8 & 13.5 & 14.2 \end{pmatrix} \cdot \%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 10.5 & 11.2 & 11.9 & 12.6 \\ 11.6 & 12.3 & 13.1 & 13.9 \\ 12.3 & 13.1 & 14 & 14.8 \end{pmatrix} \cdot \%$$

$chk_{b.3d} = "OK"$

Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 80 \cdot psf$

$Unity_{b.3g}^{T} = (4.8 \quad 4.7 \quad 4.7 \quad 4.7) \cdot \%$

$chk_{b.3j} = "OK"$

Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 1.6 & 1.5 & 1.5 & 1.4 \\ 1.8 & 1.7 & 1.6 & 1.5 \\ 1.9 & 1.8 & 1.7 & 1.6 \end{pmatrix} \cdot \%$$

$chk_{b.3j} = "OK"$

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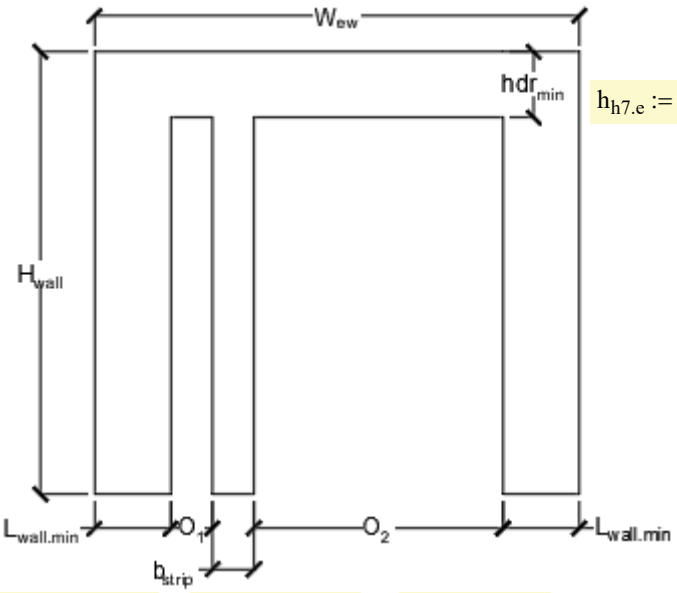
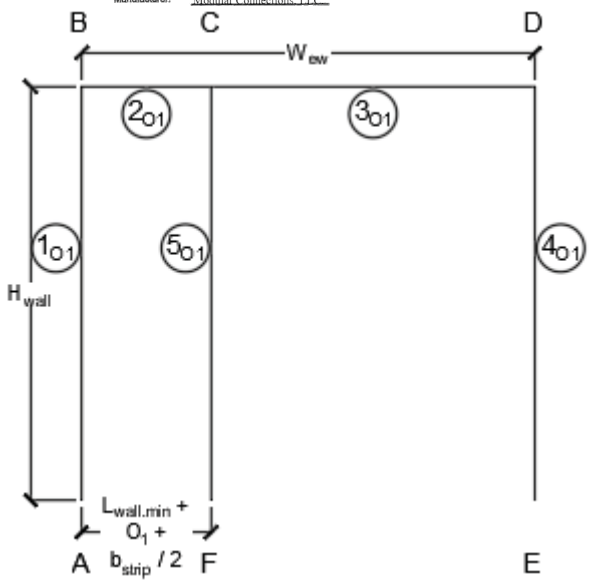
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

END WALL DESIGN



$h_{h7.e} := 19in$



$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right) \qquad Veq_{ewho}^{T} = (3.7 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip \qquad \text{Height } H_{wall_0} = 9\,ft$$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right) \qquad Veq_{ewh1}^{T} = (3.7 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip \qquad \text{Height } H_{wall_1} = 10\,ft$$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right) \qquad Veq_{ewh2}^{T} = (3.7 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip \qquad \text{Height } H_{wall_2} = 10.7\,ft$$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix} \qquad K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \dfrac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \dfrac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint C

$$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix} \qquad K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \dfrac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \dfrac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

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Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}{}^T = (6.3 \quad 6.6 \quad 7 \quad 7.4) \cdot kip \cdot ft$

$V_{L.ewh0}{}^T = (1.2 \quad 1.3 \quad 1.3 \quad 1.4) \cdot kip$

$M_{hdrh0}{}^T = (4.6 \quad 4.9 \quad 5.1 \quad 5.4) \cdot kip \cdot ft$

$V_{hdrh0}{}^T = (1.6 \quad 1.7 \quad 1.8 \quad 1.9) \cdot kip$

$M_{b.striph0}{}^T = (6.9 \quad 7.3 \quad 7.7 \quad 8.1) \cdot kip \cdot ft$

$V_{b.striph0}{}^T = (1.4 \quad 1.5 \quad 1.6 \quad 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10 ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

$DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}{}^T = (7.2 \quad 7.6 \quad 8.1 \quad 8.5) \cdot kip \cdot ft$

$V_{L.ewh1}{}^T = (1.3 \quad 1.4 \quad 1.4 \quad 1.5) \cdot kip$

$M_{hdrh1}^T = (5.7 \quad 6.1 \quad 6.4 \quad 6.8)\cdot kip\cdot ft$

$V_{hdrh1}^T = (2.6 \quad 2.7 \quad 2.9 \quad 3)\cdot kip$

$M_{b.striph1}^T = (7.7 \quad 8.2 \quad 8.6 \quad 9.1)\cdot kip\cdot ft$

$V_{b.striph1}^T = (1.4 \quad 1.5 \quad 1.6 \quad 1.1)\cdot kip$

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4\cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix}\cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119\cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4\cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh2}^T = (7.9 \quad 8.4 \quad 8.9 \quad 9.5)\cdot kip\cdot ft$

$V_{L.ewh2}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6)\cdot kip$

$M_{hdrh2}^T = (6.3 \quad 6.8 \quad 7.2 \quad 7.6)\cdot kip\cdot ft$

$V_{hdrh2}^T = (2.8 \quad 3 \quad 3.2 \quad 3.4)\cdot kip$

$M_{b.striph2}^T = (8.4 \quad 9 \quad 9.5 \quad 10.1)\cdot kip\cdot ft$

$V_{b.striph2}^T = (1.5 \quad 1.6 \quad 1.7 \quad 1.2)\cdot kip$

Reinforcement

$B_{3.1} = "\#6"$ $B_{3.1.ref} = "Wall\ Vert.\ Perim.\ Reinf."$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = "\#\ Wall\ Vert.\ Perim.\ Bars"$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.2} = "\#4"$ $B_{3.2.ref} = "Wall\ Top.\ Perim.\ Reinf."$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = "\#\ Wall\ Top\ Perim.\ Bars"$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.3} = "\#6"$ $B_{3.3.ref} = "EW\ Bot\ Perim.\ Reinf."$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = "\#\ EW\ Bot\ Perim.\ Bars"$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.4} = "\#4"$ $B_{3.4.ref} = "SW\ Bot\ Perim.\ Reinf."$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = "\#\ SW\ Bot\ Perim.\ Bars"$

$D_{3.4} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17 \cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5 \cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5 \cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{ewc} = 1.7 \cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{h.e} = 1.6 \cdot in$

$a_{strip} := \frac{A_{3.5} \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{strip} = 2.4 \cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot L_{ew.mn}$ $A_{mn.e} = 0.079 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.h.e} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.strip} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w \cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w \cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w \cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

$chk_{c.1} := \text{if}(A_{mn.e} \le \min(A_{3.1}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.1} = \text{"OK"}$

$chk_{c.2} := \text{if}(A_{mn.e} \le \min(A_{3.2}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.2} = \text{"OK"}$

$chk_{c.3} := \text{if}(A_{mn.strip} \le A_{3.5}, \text{"OK"}, \text{"NG"})$ $chk_{c.3} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{ewc} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{h.e} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{strip} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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$\phi Mn_{ewc} := \phi_{ewc} \cdot \min(A_{3.1}, A_{3.5}) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min(A_{3.2}, A_{3.5}) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref}$ = "Flexural Strength, +ve, Sect (22.3)"

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{ewc.1} := 8 \cdot \lambda \cdot \lambda_{s.ewc} \cdot \rho_{ewc}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975 \cdot kip$ $Vc_{ewc.2} = 19.387 \cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V \cdot \min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2 \cdot \left(t_w \cdot d_{ewc}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{ewc} = 3.7 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8 \cdot \lambda \cdot \lambda_{s.h.e} \cdot \rho_{h.e}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663 \cdot kip$ $Vc_{h.e.2} = 17.676 \cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V \cdot \min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.e} = 3.5 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8 \cdot \lambda \cdot \lambda_{s.strip}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347 \cdot kip$ $Vc_{strip.2} = 26.799 \cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V \cdot \min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.32 & 0.35 & 0.36 & 0.2 & 0.23 & 0.25 \\ 0.34 & 0.37 & 0.38 & 0.21 & 0.24 & 0.26 \\ 0.36 & 0.39 & 0.41 & 0.22 & 0.25 & 0.28 \\ 0.38 & 0.41 & 0.43 & 0.23 & 0.27 & 0.3 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

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$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{strip} = \begin{pmatrix} 0.17 & 0.17 & 0.18 & 0.11 & 0.12 & 0.14 \\ 0.18 & 0.18 & 0.19 & 0.12 & 0.13 & 0.14 \\ 0.19 & 0.19 & 0.2 & 0.12 & 0.14 & 0.15 \\ 0.15 & 0.14 & 0.14 & 0.13 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{hdr} = \begin{pmatrix} 0.46 & 0.73 & 0.81 & 0.17 & 0.22 & 0.24 \\ 0.48 & 0.77 & 0.87 & 0.18 & 0.23 & 0.26 \\ 0.51 & 0.82 & 0.92 & 0.19 & 0.24 & 0.27 \\ 0.54 & 0.86 & 0.98 & 0.2 & 0.26 & 0.29 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (3.7\ \ 3.9\ \ 4.2\ \ 4.4)\cdot kip$ $Vuh1^T = (3.7\ \ 3.9\ \ 4.2\ \ 4.4)\cdot kip$ $Vuh2^T = (3.8\ \ 4\ \ 4.3\ \ 4.6)\cdot kip$

Wall Plates: $No_{plates} := 1$ $V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$ $V_{u.Floor.PLh0}^T = (5.7\ \ 6.1\ \ 6.4\ \ 6.7)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$ $V_{u.Floor.PLh1}^T = (6.4\ \ 6.7\ \ 7.1\ \ 7.5)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$ $V_{u.Floor.PLh2}^T = (6.9\ \ 7.4\ \ 7.9\ \ 8.3)\cdot kip$

$A_{cv.b.3b.1}^T = (1820\ \ 1940\ \ 2060\ \ 2180)\cdot in^2$

$\rho_{b.2}^T = (0.00035\ \ 0.00033\ \ 0.00031\ \ 0.00029)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \Longrightarrow 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force

$v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.1.2h0}^T = (3.7\ \ 3.9\ \ 4.2\ \ 4.4)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.2.2h1}^T = (4.1\ \ 4.4\ \ 4.6\ \ 4.9)\cdot\dfrac{kip}{ft}$

$v_{u.5d.3.2h2} := \dfrac{V_{u.Wall.PLh2}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.3.2h2}^T = (4.5\ \ 4.8\ \ 5.1\ \ 5.4)\cdot\dfrac{kip}{ft}$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.2.2h1}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.3.2h2}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T\right)^T$$

$Hwall := \left(H_{wall_0}\ \ H_{wall_1}\ \ H_{wall_2}\right)$

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$Hwall = (9.00 \quad 10.00 \quad 10.67)\,ft$

$$Unity_{5d1.2.3} = \begin{pmatrix} 57.6 & 64 & 69.65 \\ 61.05 & 67.84 & 74.4 \\ 64.5 & 71.67 & 79.16 \\ 67.95 & 75.5 & 83.91 \end{pmatrix} \cdot \%\ L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

$chk_{5d.1} := if\left(1.0 \geq \max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4 \cdot in$ Wall Thickness $w_{plate} = 16 \cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \end{pmatrix} \quad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix} \cdot in \quad b_{eff.5d.4} := b_{eff.5d_0} \quad b_{eff.5d.5} := b_{eff.5d_1} \quad b_{eff.5d.6} := b_{eff.5d_2}$$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$ $a_{5d4} = 11.7\,ft$ $b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$ $b_{5d.4} = 8.3\,ft$ For Wall Panel Height of

$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$ $a\_b_{5d.4} = 1.4$ $l_{c.5d.4} := H_{wall_0} - t_{floor}$ $l_{c.5d.4} = 8.3\,ft$ $H_{wall_0} = 9\,ft$

$a_{5d5} := W_{ew}$ $a_{5d5} = 11.7\,ft$ $b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$ $b_{5d.5} = 9.3\,ft$ For Wall Panel Height of

$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$ $a\_b_{5d.4} = 1.4$ $l_{c.5d.5} := H_{wall_1} - t_{floor}$ $l_{c.5d.5} = 9.3\,ft$ $H_{wall_1} = 10\,ft$

$a_{5d6} := W_{ew}$ $a_{5d4} = 11.7\,ft$ $b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$ $b_{5d.6} = 10\,ft$ For Wall Panel Height of

$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$ $a\_b_{5d.6} = 1.2$ $l_{c.5d.6} := H_{wall_2} - t_{floor}$ $l_{c.5d.6} = 10\,ft$ $H_{wall_2} = 10.7ft$

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \quad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft \quad P_{5.5} := \max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right) \quad P_{5.5} = 53.3 \cdot psf$$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4} \cdot P_{5.5} \cdot \left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 15.08 \cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6} \cdot 1\,ft$ $S_{5d.4} = 32 \cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4} \cdot S_{5d.4}$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

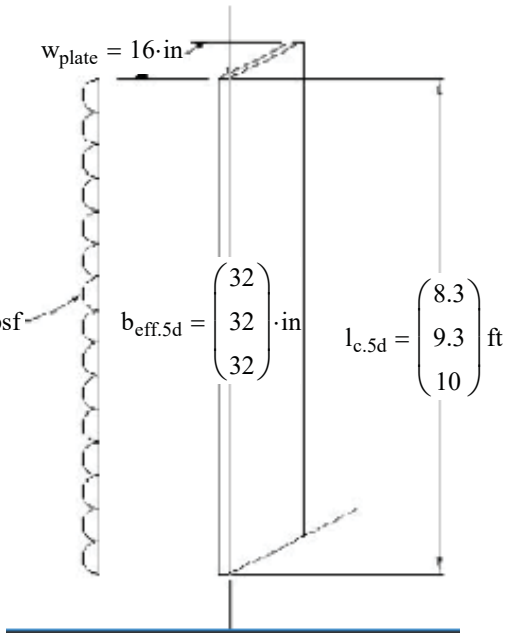
Assume only (1) end wall for resistance to lateral load $P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}{}^T = (11.4 \quad 12.1 \quad 12.8 \quad 13.5) \cdot kip$ $P_{5d.4max} := \max\left(2 \cdot V_{u.Wall.PLh0}\right)$ $P_{5d.4max} = 13.5 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}{}^T = (5.7 \quad 6.1 \quad 6.4 \quad 6.7) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \frac{\beta_{5d.5} \cdot P_{5.5} \cdot \left(l_{c.5d.5}\right)^2}{\left(t_w\right)^2} = 18.92 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \frac{\beta_{5d.6} \cdot P_{5.5} \cdot \left(l_{c.5d.6}\right)^2}{\left(t_w\right)^2} = 17.33 \cdot ksf$



$P_{5.5} = 53.3 \cdot psf$ $b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix} \cdot in$ $l_{c.5d} = \begin{pmatrix} 8.3 \\ 9.3 \\ 10 \end{pmatrix} ft$

$S_{5d.5} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$

$P_{5d.5}^T = (12.7 \quad 13.5 \quad 14.2 \quad 15) \cdot kip$

$V_{u.Floor.PLh1}^T = (6.4 \quad 6.7 \quad 7.1 \quad 7.5) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^T = (13.8 \quad 14.8 \quad 15.7 \quad 16.7) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.7 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^T = (6.9 \quad 7.4 \quad 7.9 \quad 8.3) \cdot kip$

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$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} = 224.9 \cdot kip \cdot in$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^T = (11.1 \quad 11.8 \quad 12.5 \quad 13.1) \cdot \%$

$chk_{5d.3} := if(1.0 \geq 2 \max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} = 279.8 \cdot kip \cdot in$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

$chk_{5d.4} := if\left(P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.5} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.5} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.5}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9 \cdot kip$

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^T = (14.8 \quad 15.7 \quad 16.6 \quad 17.4) \cdot \%$

$chk_{5d.5} := if(1.0 \geq 2 \max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

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$P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} = 333.2 \cdot kip \cdot in$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$ $chk_{5d.6} := if\left(P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.6} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.6}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7 \cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^T = (18.8 \quad 20 \quad 21.3 \quad 22.6) \cdot \%$

$chk_{5d.7} := if\left(1.0 \geq 2 \max\left(Unity_{5d.4}\right), "OK", "NG"\right)$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w \cdot \rho_{min.5e.1}$ $Amin_{5e.1} = 0.096 \cdot \frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^T = (1.6 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_0} = 9\ ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^T = (2.4 \quad 2.4 \quad 2.3 \quad 2.3) \cdot kip$

$V_{L.ewh0}^T = (1.2 \quad 1.3 \quad 1.3 \quad 1.4) \cdot kip$

$Unity_{5e.1.h0}^T = (50.5 \quad 54.2 \quad 58 \quad 61.9) \cdot \%$ $chk_{5e.1} := if\left(1.0 \geq \max\left(Unity_{5e.1.h0}\right), "OK", "NG"\right)$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^T = (1.6 \quad 1.6 \quad 1.6 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_1} = 10\ ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^T = (2.3 \quad 2.3 \quad 2.3 \quad 2.2) \cdot kip$

$V_{L.ewh1}^T = (1.3 \quad 1.4 \quad 1.4 \quad 1.5) \cdot kip$

$Unity_{5e.1.h1}^T = (55.5 \quad 59.7 \quad 64 \quad 68.5) \cdot \%$ $chk_{5e.2} := if\left(1.0 \geq \max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^T = (1.6 \quad 1.5 \quad 1.5 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_2} = 10.7\ ft$ $Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^T = (2.3 \quad 2.2 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh2}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^T = (58.5 \quad 63.7 \quad 69.1 \quad 74.7) \cdot \%$ $chk_{5e.3} := if\left(1.0 \geq \max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^T = (11.4 \quad 12.1 \quad 12.8 \quad 13.5) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$Mu_{5d.4} = 3.35 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \quad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.4} = 15.7 \quad d_{5e.4} := \frac{t_w}{2} \quad d_{5e.4} = 2 \cdot in \quad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'^{T}_{5e.4} = (0.29 \quad 0.28 \quad 0.26 \quad 0.25) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \quad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right] \right]$$

$P_{d.5e.4}^{T} = (269.7 \quad 272.3 \quad 274.6 \quad 276.7) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \quad Unity_{5e.4}^{T} = (4.2 \quad 4.5 \quad 4.7 \quad 4.9) \cdot \%$

$chk_{5e.4} := if(1.0 \geq \max(Unity_{5e.4}), "OK", "NG")$ $chk_{5e.4} = "OK"$

$P_{5d.5}^{T} = (12.7 \quad 13.5 \quad 14.2 \quad 15) \cdot kip$

$Mu_{5d.5} = 4.2 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \quad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.5} = 15.7 \quad d_{5e.5} := \frac{t_w}{2} \quad d_{5e.5} = 2 \cdot in \quad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'^{T}_{5e.5} = (0.33 \quad 0.31 \quad 0.3 \quad 0.28) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \quad x_{5e.5} = 159.1 \cdot kip$

$$P_{d.5e.5} := x_{5e.5} \cdot \left[\left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right] \right]$$

$P_{d.5e.5}^{T} = (263.7 \quad 266.7 \quad 269.3 \quad 271.7) \cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}} \quad Unity_{5e.5}^{T} = (4.8 \quad 5.1 \quad 5.3 \quad 5.5) \cdot \%$

$chk_{5e.5} := if(1.0 \geq \max(Unity_{5e.5}), "OK", "NG")$ $chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (13.8 \quad 14.8 \quad 15.7 \quad 16.7) \cdot kip$

$Mu_{5d.6} = 3.85 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.6} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.6}} \quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.6} = 15.7 \quad d_{5e.6} := \frac{t_w}{2} \quad d_{5e.6} = 2 \cdot in \quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$e'^{T}_{5e.6} = (0.28 \quad 0.26 \quad 0.24 \quad 0.23)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P^{T}_{d.5e.6} = (272 \quad 274.8 \quad 277.2 \quad 279.4)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity^{T}_{5e.6} = (5.1 \quad 5.4 \quad 5.7 \quad 6)\cdot\%$

$chk_{5e.6} := if\left(1.0 \geq \max\left(Unity_{5e.6}\right), "OK", "NG"\right)$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V^{T}_{u.Floor.PLh0} = (5.7 \quad 6.1 \quad 6.4 \quad 6.7)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix}735.3\\774.6\\800.8\end{pmatrix}\cdot plf$

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix}661.7\\697.1\\720.7\end{pmatrix}\cdot plf$

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Const. Type: VB
Occupancy: S2
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w^{T}_{DL.5f.1.mx} = (735.3 \quad 774.6 \quad 800.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w^{T}_{u.5f.1.DL.mx} = (661.7 \quad 697.1 \quad 720.7)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix}693.3\\732.6\\758.8\end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{(H_{wall} - t_{floor})}{3}$

$w_{5f.1} = \begin{pmatrix}2.8\\3.1\\3.3\end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix}w_{DL.5f.1.mx}\\-w_{DL.5f.1.mx}\end{pmatrix}}{No_{plates}} = \begin{bmatrix}\begin{pmatrix}735.25\\774.58\\800.81\end{pmatrix}\\\begin{pmatrix}-735.25\\-774.58\\-800.81\end{pmatrix}\end{bmatrix}\cdot plf$

$w_{up} := \frac{\left|P_{wind_3}\right|\cdot W_{ew}}{2}$ $\quad w_{up} = 311.9\cdot plf$ $\quad w_{u.up} := w_{up}$ $\quad w_{u.up} = 311.9\cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} + Embed_{5f.1}$ $\quad P_{5f.2.h0} = (8.2 \quad 8.5 \quad 8.9 \quad 9.2)\cdot kip$

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$P_{5f.2.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$ $P_{5f.2.h1} = (8.4 \;\; 8.8 \;\; 9.1 \;\; 9.4) \cdot kip$

$P_{5f.2.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$ $P_{5f.2.h2} = (8.6 \;\; 8.9 \;\; 9.3 \;\; 9.6) \cdot kip$

Tension Side

$P_{5f.3.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h0} = (-3.2 \;\; -3.6 \;\; -3.9 \;\; -4.3) \cdot kip$

$P_{5f.3.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h1} = (-3 \;\; -3.4 \;\; -3.7 \;\; -4) \cdot kip$

$P_{5f.3.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h2} = (-2.9 \;\; -3.2 \;\; -3.6 \;\; -3.9) \cdot kip$

$W_{u.5f.1} := \left| \max\left(\left| P_{W.c} \right| , \left| P_{L.c} \right| \right) \right| = 53.3 \cdot psf$ $Mu_{5f.1} := \frac{W_{u.5f.1} \cdot (H_E - 8in)^2}{8} = \begin{pmatrix} 6.3 \\ 7.8 \\ 8.9 \end{pmatrix} \cdot \frac{kip \cdot in}{ft}$

$d_{5.4} := 0.5 \cdot t_w$ $d_{5.4} = 2 \cdot in$ $x_{5f.2} := 0.00349$ Initial Guess for reinforcement ratio

$f(x_{5f.2}) := \frac{\frac{\max(Mu_{5f.1})}{\left(\frac{kip \cdot ft}{ft}\right)}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d_{5.4}}{in}\right)^2 \cdot (1 - 8.85 \cdot x_{5f.2})} - x_{5f.2}$ $\rho_{5f.2} := \frac{root(f(x_{5f.2}), x_{5f.2})}{ft}$ $\rho_{5f.2} = 0.00355 \cdot \frac{1}{ft}$

Reinforcement Ratio Required for out of plane Wind

$A_{s.req1.5f.2.} := \rho_{5f.2} \cdot 12in \cdot d_{5.4}$ $A_{s.req1.5f.2.} = 0.085 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for out of plane Wind

$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9 \cdot f_y \cdot \frac{W_{ew}}{2}} \right)$

$A_{s.req2.5f.2} = 0.01358 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for Tension Force from Uplift

$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2}$ $A_{req.5f.2} = 0.099 \cdot \frac{in^2}{ft}$ Total Area of Reinforcement Required

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ Area of Steel Provided

$chk_{5.8} := if(A_{req.5f.2} \le A_{s.4a}, "OK", "NG")$ $chk_{5.8} = "OK"$

Check P-Δ Effects

$E_{c.spec} = 2991 \cdot ksi$ $I_{g.5g.1} := \frac{1}{12} \cdot 12in \cdot (t_w)^3$ $I_{g.5g.1} = 64 \cdot in^4$

$P_{cr.5g.1} := \frac{\pi^2 \cdot E_{c.spec} \cdot I_{g.5g.1}}{H_E^2}$ $P_{cr.5g.1} = \begin{pmatrix} 143.5 \\ 117.6 \\ 104.1 \end{pmatrix} \cdot kip$ $Ma_{g.2} := \frac{\left| \max\left(\left| P_{W.c} \right| , \left| P_{L.c} \right| \right) \right| \cdot (H_{wall})^2}{8} \cdot 12in$ $Ma_{g.2} = \begin{pmatrix} 539.3 \\ 665.9 \\ 757.6 \end{pmatrix} \cdot lb \cdot ft$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$Mb_{5g.2} := Ma_{g.2} \quad Mb_{5g.2} = \begin{pmatrix} 0.5 \\ 0.7 \\ 0.8 \end{pmatrix} \cdot kip \cdot ft \qquad C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}} \qquad C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}} \qquad \delta_{ns.5.6} = \begin{pmatrix} 1.2 \\ 1.2 \\ 1.2 \end{pmatrix}$$

$$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix} \qquad Ma_{g.3} = \begin{pmatrix} 622.6 \\ 795.7 \\ 929 \end{pmatrix} \cdot lb \cdot ft \qquad f_r = 427.6 \cdot psi \qquad M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}} \qquad M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking\ Occurs", "Cracking\ Doesn't\ Occur"\right)$ $\qquad chk_{5.9} = "Cracking\ Doesn't\ Occur"$

$$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}} \qquad \Delta_{5g.1} = \begin{pmatrix} 0.18 \\ 0.23 \\ 0.27 \end{pmatrix} \cdot in \qquad Mu_{5.5} := Mu_{5f.1} \cdot ft \qquad Mu_{5.5} = \begin{pmatrix} 6.3 \\ 7.8 \\ 8.9 \end{pmatrix} \cdot kip \cdot in$$

$$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$$

$$Mu_{5.6} = (10.8 \quad 10.9 \quad 11 \quad 11.1) \cdot kip \cdot in$$

$$P_{5.8} := Embed_{5f.1}$$

$$P_{5.8} = (5.7 \quad 6.1 \quad 6.4 \quad 6.7) \cdot kip$$

$$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}} \qquad e`_{5.6} = (1.9 \quad 1.8 \quad 1.7 \quad 1.7) \cdot in$$

$$u_{5.} := \frac{f_y}{0.85 \cdot f`_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f`_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

$$\rho_{5.8} := \frac{A_{s.4a}}{\max\left(w_{5f.1}\right) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}} + \sqrt{\left(1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max\left(e`_{5.6}\right) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 230.3 \\ 257.9 \\ 276.3 \end{pmatrix} \cdot kip$$

$chk_{5.10} := if\left(\max\left(P_{5.8}\right) \le \min\left(\phi P_{5.6}\right), "OK", "NG"\right)$ $\qquad chk_{5.10} = "OK"$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 37.6 \\ 37.5 \\ 37.4 \\ 37.3 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 30 ' "} \\ \text{"Min Flr Uniform Weight, for L = 32 ' "} \\ \text{"Min Flr Uniform Weight, for L = 34 ' "} \\ \text{"Min Flr Uniform Weight, for L = 36 ' "} \end{pmatrix}$$

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$$wg_{f.mx} = \begin{pmatrix} 41.6 \\ 41.5 \\ 41.4 \\ 41.3 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$$ Wall Weight

$$w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf$$ Wall Misc. Weight

$DL_{r.mx} = 63.5 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 56.5 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -53.48 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 46 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.157$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

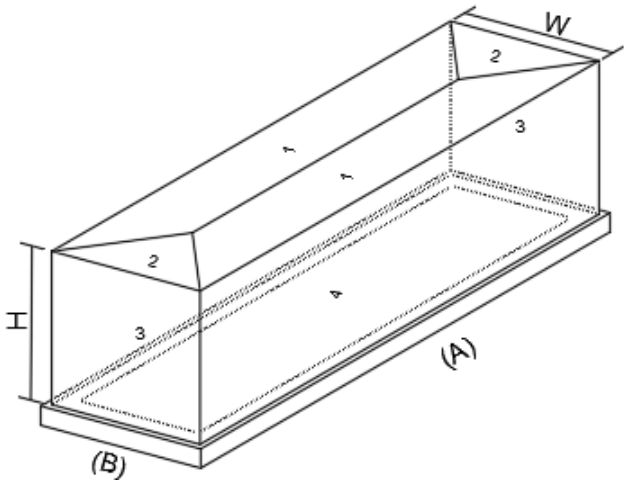
$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.126$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (364 \quad 388 \quad 412 \quad 436)\ ft^2$

$A_f^T = (320.99 \quad 342.9 \quad 364.82 \quad 386.74)\ ft^2$

$L_{wall}^T = (81.92 \quad 85.92 \quad 89.92 \quad 93.92)\ ft$



$$DL_{A.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) + \frac{W_f}{2} \cdot (wgt_{flr}) \ldots + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right) \dots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 57.4\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right) \dots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 50.16\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor} \dots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right] \dots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}\right] \qquad \begin{matrix} z_{mx} = 67.38\cdot in \\ z_{mx} = 5.61\cdot ft\end{matrix}$$

$ftg_{exp} := 36in$ Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.73\,ft$ Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\,ft$ Moment Arm for Floor Level

$DL_{A.f0.mx}^{T} = (1088.8 \quad 1088.3 \quad 1087.8 \quad 1087.5)\cdot plf$

$DL_{A.f0.mn}^{T} = (1040 \quad 1030.5 \quad 1022.2 \quad 1014.8)\cdot plf$

$DL_{B.f0.mx}^{T} = (834.6 \quad 834.1 \quad 833.7 \quad 833.3)\cdot plf$

$DL_{B.f0.mn}^{T} = (813.9 \quad 804.4 \quad 796 \quad 788.6)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1551 \quad 1551 \quad 1550 \quad 1550)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (1040 \quad 1031 \quad 1022 \quad 1015)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (837 \quad 837 \quad 837 \quad 837)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (586 \quad 586 \quad 586 \quad 585)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C}$ $\qquad E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$E_{O.A.E.f0.C}^{T} = (225.3 \;\; 225.2 \;\; 225.1 \;\; 225.1)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-225.3 \;\; -225.2 \;\; -225.1 \;\; -225.1)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (45.5 \;\; 42.6 \;\; 40 \;\; 37.7)\cdot plf$

$E_{O.B.E.f0.T}^{T} = (-45.5 \;\; -42.6 \;\; -40 \;\; -37.7)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (215.3 \;\; 215.2 \;\; 215.2 \;\; 215.1)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-215.3 \;\; -215.2 \;\; -215.2 \;\; -215.1)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (44.2 \;\; 41.4 \;\; 38.9 \;\; 36.7)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-44.2 \;\; -41.4 \;\; -38.9 \;\; -36.7)\cdot plf$

$E_{O.A.E.f2.C}^{T} = (205.4 \;\; 205.3 \;\; 205.2 \;\; 205.2)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-205.4 \;\; -205.3 \;\; -205.2 \;\; -205.2)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (43 \;\; 40.2 \;\; 37.8 \;\; 35.7)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-43 \;\; -40.2 \;\; -37.8 \;\; -35.7)\cdot plf$

$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$

$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$

$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$

$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$

$E_{8p.A.f0}^{T} = (664.6 \;\; 664.4 \;\; 664.2 \;\; 664)\cdot plf$

$E_{8n.A.f0}^{T} = (-192.5 \;\; -192.8 \;\; -193 \;\; -193.1)\cdot plf$

$E_{8p.B.f0}^{T} = (282.5 \;\; 279.5 \;\; 276.9 \;\; 274.6)\cdot plf$

$E_{8n.B.f0}^{T} = (-27 \;\; -24.1 \;\; -21.6 \;\; -19.3)\cdot plf$

$E_{8p.A.f1}^{T} = (635.2 \;\; 635 \;\; 634.8 \;\; 634.7)\cdot plf$

$E_{8p.B.f1}^{T} = (274.8 \;\; 271.9 \;\; 269.4 \;\; 267.1)\cdot plf$

$E_{8n.A.f1}^{T} = (-170.7 \;\; -170.6 \;\; -170.6 \;\; -170.5)\cdot plf$

$E_{8n.B.f1}^{T} = (-19.3 \;\; -16.5 \;\; -14 \;\; -11.8)\cdot plf$

$E_{8p.B.f2}^{T} = (267.1 \;\; 264.3 \;\; 261.8 \;\; 259.6)\cdot plf$

$E_{8n.A.f2}^{T} = (-162.9 \;\; -162.8 \;\; -162.8 \;\; -162.7)\cdot plf$

$E_{8p.A.f2}^{T} = (605.9 \;\; 605.7 \;\; 605.5 \;\; 605.3)\cdot plf$

$E_{8n.B.f2}^{T} = (-18.8 \;\; -16 \;\; -13.6 \;\; -11.4)\cdot plf$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$LL_{r.A.} := \frac{W_r}{2} \cdot (w_{llr})$ $LL_{r.A.} = 480 \cdot plf$ $LL_{r.B.} := \frac{W_r}{6} \cdot (w_{llr})$ $LL_{r.B.} = 160 \cdot plf$

$LL_{A.f0} := \frac{W_f}{2} \cdot (w_{llf.0})$ $LL_{A.f0} = 1369.79 \cdot plf$ $LL_{B.f0} := s_{t.rib} \cdot (w_{llf.0})$ $LL_{B.f0} = 500 \cdot plf$

$LL_{A.f1} := \frac{W_f}{2} \cdot (w_{llf.1})$ $LL_{A.f1} = 1095.83 \cdot plf$ $LL_{B.f1} := s_{t.rib} \cdot (w_{llf.1})$ $LL_{B.f1} = 400 \cdot plf$

$LL_{A.f2} := \frac{W_f}{2} \cdot (w_{llf.2})$ $LL_{A.f2} = 821.88 \cdot plf$ $LL_{B.f2} := s_{t.rib} \cdot (w_{llf.2})$ $LL_{B.f2} = 300 \cdot plf$

$SL_A := \frac{W_r}{2} \cdot (p_f)$ $SL_A = 190.51 \cdot plf$ $SL_B := \frac{W_r}{6} \cdot (p_f)$ $SL_B = 63.5 \cdot plf$

$W_{up.A} := \frac{W_r}{2} \cdot P_{wind_3}$ $W_{up.A} = -320.86 \cdot plf$ $W_{up.B} := \frac{W_r}{6} \cdot P_{wind_3}$ $W_{up.B} = -106.95 \cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat} \cdot H_{wall} \cdot \left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $W_{OT.A.C} = 618.5 \cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat} \cdot H_{wall} \cdot \left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $W_{OT.B.C} = 231.4 \cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $W_{OT.A.T} = -618.5 \cdot plf$ $W_{OT.B.T} := -W_{OT.B.C}$ $W_{OT.B.T} = -231.4 \cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $W_{A.mx} = 297.6 \cdot plf$ $W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $W_{B.mx} = 124.4 \cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $W_{A.mn} = -939.4 \cdot plf$ $W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $W_{B.mn} = -338.3 \cdot plf$

b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \qquad Load_{f0} = \begin{pmatrix} 1088.76 \\ 1014.76 \\ 1369.79 \\ 0 \\ 480 \\ 190.51 \\ 297.65 \\ -939.36 \\ 664.57 \\ -193.14 \end{pmatrix} \cdot plf$$

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$$\text{ASD} = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f0} := asd \cdot Load_{f0} \quad LC_{8.f0} = \begin{pmatrix} 2459 \\ 1569 \\ 2596 \\ 1267 \\ 525 \\ 2579 \\ 1652 \\ 787 \\ 45 \\ 1554 \\ 954 \\ 2608 \\ 2158 \\ 1074 \\ 474 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 12.52 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 13 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f0} := \max(14in, w_{8.f0.min})$ $\quad Fdn_{W.f0} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 14 \cdot in$

consider $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \quad Load_{f1} = \begin{pmatrix} 1088.76 \\ 1014.76 \\ 1095.83 \\ 0 \\ 480 \\ 190.51 \\ 297.65 \\ -939.36 \\ 635.23 \\ -170.67 \end{pmatrix} \cdot plf$$

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$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f1} := asd \cdot Load_{f1} \quad LC_{8.f1} = \begin{pmatrix} 2185 \\ 1569 \\ 2391 \\ 1267 \\ 525 \\ 2374 \\ 1446 \\ 787 \\ 45 \\ 1533 \\ 969 \\ 2387 \\ 1964 \\ 1054 \\ 489 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 11.48 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 12 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f1} := \max(14in, w_{8.f1.min})$ $\quad Fdn_{W.f1} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

consider $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \quad Load_{f2} = \begin{pmatrix} 1088.76 \\ 1014.76 \\ 821.88 \\ 0 \\ 480 \\ 190.51 \\ 297.65 \\ -939.36 \\ 605.88 \\ -162.88 \end{pmatrix} \cdot plf$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 10.49 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 11 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f2} := \max(12in, w_{8.f2.min})$ $\quad Fdn_{W.f2} = 12 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 12 \cdot in$

c. Slab on Grade Foundation

1.) Criteria: use slab on grade. $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $\quad Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.long} \qquad A_{s.long.prov} = 0.39 \cdot in^2$$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if\left(A_{s.long.prov} \geq A_{s.long}, "OK", "NG"\right)$ $chk_{8.1} = "OK"$

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp} \qquad A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4 \qquad Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.temp}$$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if\left(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG"\right)$ $chk_{8.2} = "OK"$

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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 56.542 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 63.5 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 80 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 46 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 35.8 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 32.3 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 53.5 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 3.01 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 3.012 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 9.295 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.021 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.021 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.398 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.398 \cdot klf$

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SEISMIC LOAD PARAMETERS: *For worst case seismic load in E-W direction, look at longest, widest shelter.*

Seismic Coefficient

$C_s = 0.157$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 36.42\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 33.37\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 34.82\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 29.38\cdot kip$

Total Seismic Shear at Roof

$$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$$ $V_{smaxW\_rf} = 8.47\cdot kip$

$$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$$ $V_{sminW\_rf} = 7.561\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$$ $v_{EW\_smaxW} = 0.141\cdot klf$

$$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$$ $v_{EW\_sminW} = 0.105\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$$ $v_{NS\_smaxW} = 0.363\cdot klf$

$$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$$ $v_{NS\_sminW} = 0.324\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$$ Modification Factor for Eccentric Loading

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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k_c — Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ — Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ — Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ — Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} — Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} — Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ — Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ — Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ — Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix}$ = ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ — Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.038 \cdot klf$ — Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.363 \cdot klf$ — Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.16 \cdot klf$ — Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.212 \cdot klf$ — Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.094 \cdot klf$ — Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.398 \cdot klf$ — Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 7$

$$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1} \qquad s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$	$Vu_{o.s.REW} = 0.021\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$	$Vu_{i.s.REW} = 0.659\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$	$Vu_{o.w.REW} = 0.171\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$	$Vu_{i.w.REW} = 0.723\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.REW} := F_{7.1}\cdot max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$	$Tu_{T.REW} = 0\cdot kip$	0.9DL + 1.0W	T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$	$Vu_{o.s.EWR} = 0.021\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$	$Vu_{i.s.EWR} = 0.682\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.EWR} := F_{7.2}\cdot max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$	$Vu_{o.w.EWR} = 0.177\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$	$Vu_{i.w.EWR} = 0.749\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$	$Tu_{T.EWR} = 0.177\cdot kip$	0.9DL + 1.0W	T, roof uplift

▶ Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 5\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 5 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 112 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 5.72 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 5 \cdot in$ $h_{min.7.1a} = 5 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 134 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 3.76 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 4.287 \\ 2.816 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min((0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1}))$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.239 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.297 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.026$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.29 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.365 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.1} = 0.2 \cdot in$$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.2} = 0.231 \cdot in$$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2} \qquad l_{w\_Rqrd.REW.3} = 2.5 \cdot in$$

$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right)$ $l_{w\_reqd.REW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.038\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.141\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.321\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.381\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.188\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.021\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 48\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.3} := \text{if}(s_{L.7.3} = 0in, s_{w.7.3}, \text{if}(s_{w.7.3} = 0in, s_{L.7.3}, \min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := \text{if}(s_{L.7.4} = 0in, s_{w.7.4}, \text{if}(s_{w.7.4} = 0in, s_{L.7.4}, \min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.178\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.198\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 0.878\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.098\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.178\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.198\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 0.878\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.098\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.141 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.327 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.136 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.308 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.078\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.259\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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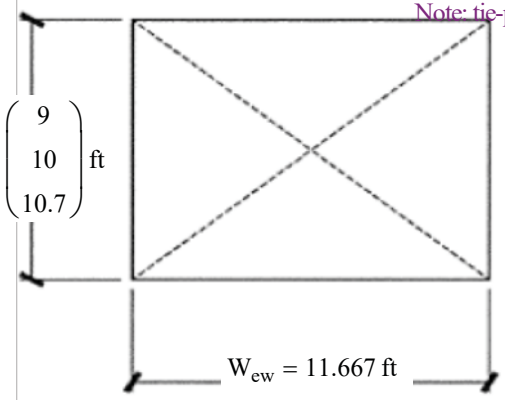
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 7$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}$ ft

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (13.667 \quad 15.667 \quad 17)\cdot in$ $s_{SWEW}^{T} = (13.667 \quad 15.667 \quad 17)\cdot in$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6\,ft$ Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 5.275\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{ew}\cdot L_{ew}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (0.542 \quad 0.508 \quad 0.479 \quad 0.452)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.007\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 0.888\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10\,ft$

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

$M_{ot\_w.EWSW}^{T} = (59.9 \quad 62.8 \quad 65.8 \quad 68.8)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r + \frac{\max\left[\left(\omega_{wall}\right)\cdot\left[\max\left(H_{wall}\right)\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (75.2 \quad 78.4 \quad 81.5 \quad 84.7)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (93.1 \quad 98.3 \quad 103.6 \quad 108.9)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (-23.9 \quad -25.67 \quad -27.44 \quad -29.21)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (-17.83 \quad -19.96 \quad -22.09 \quad -24.23)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max\left(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net}\right)$ $M_{ot.EWSW} = 0\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := \text{if}\left(s_{L.7.5} = 0in, s_{w.7.5}, \text{if}\left(s_{w.7.5} = 0in, s_{L.7.5}, \min\left(s_{L.7.5}, s_{w.7.5}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := \text{if}\left(s_{L.7.6} = 0in, s_{w.7.6}, \text{if}\left(s_{w.7.6} = 0in, s_{L.7.6}, \min\left(s_{L.7.6}, s_{w.7.6}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.334\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.334\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Occupancy: S2
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wal (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.334 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.334 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.078$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.115$ < 1.0

$Result_{EWSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW}$ = "not required" < 1.2

$chk_{6.EWSW}$ = "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.079$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.207$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.139 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.139 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right)$ $l_{w\_reqd.EWSW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.68 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.247 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.019 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.363 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.398 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.165 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.094 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\,ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$M_{ot.s.FEW}^{T} = (70.7 \quad 75.3 \quad 80 \quad 84.7)\cdot kip\cdot ft$

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix}0.5\cdot(L_0)^2\\0.5\cdot(L_1)^2\\0.5\cdot(L_2)^2\\0.5\cdot(L_3)^2\end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$$

OT Moment, resisting

$M_{res.FEW}^{T} = (941.2 \quad 1004 \quad 1066.7 \quad 1129.5)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-447.3 \quad -453.4 \quad -456.4 \quad -456.1)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = \left(-870.6 \quad -928.6 \quad -986.7 \quad -1\times 10^{3}\right)\cdot kip\cdot ft$

$M_{ot.FEW} := \max\left(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net}\right)$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right] \qquad v_{ot.w.FEW} = 0\cdot kip$$

$$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right) \qquad v_{ot.s.FEW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor $\qquad$ $Plate_{7.8}$ = "#2" at inside face of end wall

No. of Plates & maximum plate spacing $\quad No_{pl.FEW} = 7$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \qquad s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1}$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$s_{F\_EW} = 15.92\cdot in$ $\qquad s_{EW\_F} = 17.33\cdot in$

$s_{7.7} := \text{if}\left(s_{L.7.7} = 0in, s_{w.7.7}, \text{if}\left(s_{w.7.7} = 0in, s_{L.7.7}, \min\left(s_{L.7.7}, s_{w.7.7}\right)\right)\right) = 3\cdot in$ $\qquad s_{7.7} = 3\cdot in$

$F_{7.7} := 0.5 + \frac{0.5s_{7.7} + 2e_{PL.F\_EW}}{s_{7.7}}$ $\qquad F_{7.7} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.7

$s_{7.8} := \text{if}\left(s_{L.7.8} = 0in, s_{w.7.8}, \text{if}\left(s_{w.7.8} = 0in, s_{L.7.8}, \min\left(s_{L.7.8}, s_{w.7.8}\right)\right)\right) = 3\cdot in$ $\qquad s_{7.7} = 3\cdot in$

$F_{7.8} := 0.5 + \frac{0.5s_{7.8} + 2e_{PL.F\_EW}}{s_{7.8}}$ $\qquad F_{7.8} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.8

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right)$$ $Vu_{i.s.FEW} = 1.033\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{i.w.FEW} = 0.775\cdot kip$ 1.0W in-plane V, wind roof diaphragm

$$Tu_{po.s1.FEW} := F_{7.7}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right)$$ $Tu_{po.s1.FEW} = 0.01\cdot kip$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Tu_{po.w.FEW} = 0.366\cdot kip$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{o.9dl.FEW} = 1.189\cdot kip$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7}\cdot\frac{v_{ot.w.FEW}}{No_{pl.FEW}}$$ $V_{ot.w.FEW} = 0\cdot kip$ 1.0W

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot.s.FEW} := F_{7.7}\cdot\frac{v_{ot.s.FEW}}{No_{pl.FEW}}$$ $V_{ot.s.FEW} = 0\cdot kip$ 1.0E

$$Vu_{o.wup.FEW} := F_{7.7}\cdot\max\left[\left(1.0v_{up.w.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0\cdot kip\right]$$

$Vu_{o.wup.FEW} = 0\cdot kip$ 0.9D+1.0W

$$Vu_{o.sup.FEW} := F_{7.7}\cdot\max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0\cdot kip\right)$$

Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.sup.FEW} = 0\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7}\cdot\left[1.2\left(v_{DL.FEW}\right) + 1.6v_{LL.FEW}\right]\cdot\frac{W_{ew}}{No_{pl.FEW}}$$

$Vu_{o.12D.16L.FEW} = 2.354\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Endwall (Floor Wall Plates)

$$Tu_{po.w.EWF} := F_{7.8}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Tu_{po.w.EWF} = 0.366\cdot kip$ pullout, wind suction at bot of wall and bot half of two sides of wall

$$Tu_{po.s2.EWF} := F_{7.8}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right)$$ $Tu_{po.s2.EWF} = 0.01\cdot kip$ 0.3E pullout, seismic

$$Vu_{i.s.EWF} := F_{7.8}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right)$$ $Vu_{i.s.EWF} = 1.033\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w2.EWF} := F_{7.8}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{i.w2.EWF} = 0.775\cdot kip$ 1.0W in-plane V, wind roof diaphragm

$$Vu_{o.9dl.EWF} := F_{7.8}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{o.9dl.EWF} = 1.189\cdot kip$ 0.9D Shear from DL only

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$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - \left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 1.067 \cdot kip$ 0.9D+1.0W

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[\left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

$Vu_{o.sdn.EWF} := F_{7.8} \cdot \left(Vu_{o.9dl.FEW} + V_{ot.s.FEW}\right)$ $Vu_{o.sdn.EWF} = 1.388 \cdot kip$ 0.9D+1.0E

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6 v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 2.354 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$

$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75 \cdot V_{cbg\_op.7.7} \\ 0.75 \cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_EW} = \min\left(\left(0.75 \cdot N_{sa.7.7} \quad 0.75 \cdot N_{cbg.7.7} \quad 0.7 \cdot N_{pn.7.7}\right)\right)$ $\phi P_{pl.F\_EW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041 \quad \text{or} \quad \left[\frac{Vu_{o.wup.FEW}}{0.75 \cdot V_{cbg\_op.7.7}} + \frac{Vu_{i.w.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.158 < 1.0$$

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041 \quad \text{or} \quad \left[\frac{Vu_{o.sup.FEW}}{\left(0.75 \cdot V_{cbg\_op.7.7}\right)} + \frac{Vu_{i.s.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.211 < 1.0$$

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$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.692 \quad < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted"

$IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $V_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719\cdot in$ $h_{min.7.8} = 4\cdot in$ $c_{a1\_op.7.8} = 6.5\cdot in$ $c_{a2\_op.7.8} = 10.5\cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90\cdot in^2$ $A_{Vco\_op.7.8} = 190\cdot in^2$ $e'_{V.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0\cdot in$ $s2_{7.8a} = 3\cdot in$ $l_{e.7.8} = 1.719\cdot in$ $h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$ $h_{min.7.8a} = 4\cdot in$

$c_{a1\_ip.7.8} = 13.5\cdot in$ $c_{a2\_ip.7.8} = 6.5\cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107\cdot in^2$ $A_{Vco\_ip.7.8} = 820\cdot in^2$ $e'_{V.7.8a} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3\cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74\cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8}\cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74\cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75\cdot V_{cbg\_op.7.8} \\ 0.75\cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix}\cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\left(0.75\cdot N_{sa.7.8} \quad 0.75\cdot N_{cbg.7.8} \quad 0.7\cdot N_{pn.7.8}\right)\right)$ $\phi P_{pl.EW\_F} = 4.305\cdot kip$

Plate Summary

$$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.003 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.863 \quad < 1.0$$

$$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.085 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.655 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75\cdot V_{cbg\_op.7.8}}\right) = 0.686 \quad < 1.0$$

$Result_{EW\_F} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F} =$ "not required" < 1.2

$chk_{6.EW\_F} =$ "OK"

Weld Between $Plate_{7.7} =$ "#1B" and $Plate_{7.8} =$ "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.1} = 0.251\cdot in$$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.2} = 0.303\cdot in$$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.3} = 0.401\cdot in$$

MODULAR CONNECTIONS, LLC.
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^{2} + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^{2} + Vu_{i.s.EWF}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.507\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.507\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$l_{w\_reqd.EWSW} = 2.5\cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

****

****

f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $\quad w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $\quad v_{dl.FSW} = 0.865\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $\quad v_{ll.FSW} = 0.48\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $\quad t_{o.s.FSW} = 0.038\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $\quad v_{i.s.FSW} = 0.141\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $\quad v_{i.w.SW} = 0.0209\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $\quad v_{up.w.FSW} = 0.321\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $\quad t_{po.w.FSW} = 0.172\cdot klf$ Wind load suction on bottom of side wall

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^{T} = (70.7 \quad 75.3 \quad 80 \quad 84.7)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $\quad M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^{T} = (941.2 \quad 1004 \quad 1066.7 \quad 1129.5)\cdot kip\cdot ft$

$$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.w.FSW} = 0\cdot kip$$

$$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.s.FSW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9}$ = "#1B" at top of floor

$Plate_{7.10}$ = "#2" at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 18\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$$s_{7.9} := \text{if}\left(s_{L.7.9} = 0in, s_{w.7.9}, \text{if}\left(s_{w.7.9} = 0in, s_{L.7.9}, \min\left(s_{L.7.9}, s_{w.7.9}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}} \qquad F_{7.9} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.9

$$s_{7.10} := \text{if}\left(s_{L.7.10} = 0in, s_{w.7.10}, \text{if}\left(s_{w.7.10} = 0in, s_{L.7.10}, \min\left(s_{L.7.10}, s_{w.7.10}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}} \qquad F_{7.10} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.FSW} = 0.301\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.FSW} = 0.067\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.FSW} = 0.074\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.FSW} = 0.037\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.FSW} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.FSW} = 1.167\cdot kip$ 0.9D Shear from DL only

$$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}} \qquad V_{ot\_w.FSW} = 0\cdot kip$$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}} \qquad V_{ot\_s.FSW} = 0\cdot kip$$

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\begin{bmatrix}\left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, \left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, 0\cdot kip \\ +V_{ot\_w.FSW} \qquad\qquad +-V_{ot\_w.FSW}\end{bmatrix}$$

$Vu_{o.w.FSW} = 1.924\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$$

$Vu_{o.s.FSW} = 1.362\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$$

$Vu_{o.12D.16L.FSW} = 3.16\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.301\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.067\cdot kip$ 0.3W T, pullout seismic

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$Vu_{i.s.SWF} := F_{7.10} \cdot 0.3 v_{i.s.FSW} \cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.074 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10} \cdot 1.0 v_{i.w.SW} \cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.037 \cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := (0.9 v_{dl.FSW}) \cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.167 \cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0 \cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0 \cdot kip$

$$Vu_{o.w.SWF} := F_{7.10} \cdot \max\left[\begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, \begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +-V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, 0 \cdot kip\right]$$

$Vu_{o.w.SWF} = 1.924 \cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.SWF} := F_{7.10} \cdot \max(V_{ot\_s.SWF} - 0.9 v_{dl.FSW} \cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9 v_{dl.FSW} \cdot s_{max.FSW}, 0 \cdot kip)$

$Vu_{o.s.SWF} = 1.362 \cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.SWF} := F_{7.10} \cdot [1.2(v_{dl.FSW}) + 1.6 v_{ll.FSW}] \cdot s_{max.FSW}$

$Vu_{o.12D.16L.SWF} = 3.16 \cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5 \cdot in$ $w_{pl.7.9} = 5 \cdot in$ $x_{Le.7.9} = 1 \cdot in$ $x_{we.7.9} = 4 \cdot in$ $t_{pl.7.9} = 0.313 \cdot in$

$h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0 \cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5 \cdot in$ $A_{SE.7.9} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $N_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.034 \quad \text{or} \quad \left[\frac{Vu_{o.w.FSW}}{\left(0.75 \cdot V_{cbg\_op.7.9}\right)} + \frac{Vu_{i.w.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.574 < 1.0$$

$$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.007 \quad \text{or} \quad \left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.417 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.929 < 1.0$$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.07 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.574 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.015 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.427 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.918 < 1.0$$

$Result_{SWF} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.SWF} = "not required"$ < 1.2

$chk_{6.SWF} = "OK"$

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.1} = 0.571 \cdot in$$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.2} = 0.4 \cdot in$$

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.571 \cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.4 \cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.204 \cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min\left(w_{pl.7.9}, w_{pl.7.10}\right)}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5 \cdot in$$

$l_{w\_reqd.FSW} := \max\left(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6}\right)$ $l_{w\_reqd.FSW} = 2.5 \cdot in$

MODULAR CONNECTIONS, LLC.
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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $\quad wgt_r = 56.542\cdot psf$ — Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $\quad wgt_w = 39.333\cdot psf$ — Wall Dead Load

$wgt_{flr}^{T} = (37.56 \quad 37.47 \quad 37.39 \quad 37.32)\cdot psf$ — Floor Dead Load

$W_{lat} = 45.986\cdot psf$ — Wind Load

$w_{llf.0} = 250\cdot psf$ $\quad w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200\cdot psf$ $\quad w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150\cdot psf$ $\quad w_{llf.2.ref}$ = "Floor Live Load Option 2"

$A_r := L_r\cdot W_r$ $\quad A_r^{T} = (364 \quad 388 \quad 412 \quad 436)\,ft^2$ — Area of Roof/Floor

$A_f := L_f\cdot W_f$ $\quad A_f^{T} = (321 \quad 342.9 \quad 364.8 \quad 386.7)\,ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$

Allowable Resisting Moment

$M_R^{T} = (519 \quad 531.3 \quad 543.6 \quad 555.9)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$

Overturning Moment due to Wind

$M_{OT.wind}^{T} = (121.5 \quad 129.6 \quad 137.7 \quad 145.8)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^{T} = (182.2 \quad 194.4 \quad 206.5 \quad 218.7)\cdot kip\cdot ft$

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\left[\begin{matrix} L_r\cdot W_r\cdot(wgt_r)\cdot\max(H_E) + L_r\cdot W_r\cdot(0.25w_{llr})\cdot\max(H_E) \ldots \\ +\left(2L_f\cdot\max(H_E) + 2W_{ew}\cdot\max(H_E)\right)\cdot\frac{wgt_w\cdot\max(H_E)}{2} \ldots \\ + L_f\cdot W_f\cdot wgt_{flr}\cdot\frac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot(0.25w_{llf.0})\cdot t_{floor} \end{matrix}\right]$$

Overturning Moment due to seismic

$M_{OT.seismic}^{T} = (60.895 \quad 64.36 \quad 67.825 \quad 71.289)\cdot kip\cdot ft$

$1.5\,M_{OT.seismic}^{T} = (91.342 \quad 96.54 \quad 101.737 \quad 106.934)\cdot kip\cdot ft$

$M_{ot.w.net} := (1.5M_{OT.wind} - M_R)$ $\quad M_{ot.s.net} := (1.5M_{OT.seismic} - M_R)$

$M_{ot.w.net}^{T} = (-336.7 \quad -336.9 \quad -337 \quad -337.2)\cdot kip\cdot ft$

$M_{ot.s.net}^{T} = (-427.6 \quad -434.7 \quad -441.8 \quad -448.9)\cdot kip\cdot ft$

$chk_{S7.15} := \text{if}(\max(M_{ot.w.net}, M_{ot.s.net}) \geq 0, \text{"NG"}, \text{"OK"})$ $\quad chk_{S7.15}$ = "OK"

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Sliding

$$WT := \begin{pmatrix} L_r \cdot W_r \cdot wgt_r + L_f \cdot \min(H_E) \cdot wgt_w \ldots \\ + 2W_{ew} \cdot \min(H_E) \cdot wgt_w + 2L_f \cdot W_f \cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^T = (146.331 \quad 148.44 \quad 150.55 \quad 152.659) \cdot kip$$

$$WT_{max} := \max\begin{bmatrix} A_r \cdot (wgt_r + w_{misc.r}) + A_f \cdot (wgt_{flr} + w_{misc.f}) \ldots \\ + 2L_f \cdot \max(H_E) \cdot (wgt_w + w_{misc.w}) \ldots \\ + 2 \cdot W_{ew} \cdot \max(H_E) \cdot wgt_w + [A_r \cdot w_{llr} + A_r \cdot (w_{llf.0})] \end{bmatrix}$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 276.461 \cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in \cdot 4in$ $A_{pad} = 16 \cdot in^2$

$S_{bp} := 3ft$

Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad} = 4.872 \cdot kip$

Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 304.5 \cdot psi$

Compressive Stress per Pad

$F_{pad} := 800psi$

Allowable Stress per pad

$F_P := 0.7 \cdot 0.85 f'_{c.fdn}$

$F_P = 1785.1 \cdot psi$

Allowable Stress of Fdn Concrete

$chk_{S7.16} := \text{if}(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, \text{"OK"}, \text{"NG"})$

$chk_{S7.16} = \text{"OK"}$

$$wt_{pad.big} := \max\left[\frac{\max(WT_{max})}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad.big} = 8.823 \cdot kip$

Weight per Bearing Pad

$$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$$

$f_{pad.big} = 551.5 \cdot psi$

Compressive Stress per Pad

$chk_{S7.17} := \text{if}(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, \text{"OK"}, \text{"NG"})$

$chk_{S7.17} = \text{"OK"}$

$\mu := 0.25$

Coefficient of Friction

Wind $V_{net.w.sl} := \max\begin{bmatrix} 0kip, \begin{bmatrix} \mu \cdot [-(WT) + 1.5 |P_{wind_3}| \cdot (A_r)] \ldots \\ + 1.5W_{lat} L \cdot \max(H_E) \end{bmatrix} \end{bmatrix}$

$V_{net.w.sl} = 0 \cdot kip$

Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := \max\begin{bmatrix} 0kip, 1.5 \cdot \begin{bmatrix} 0.7C_s \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} + -\mu \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} \end{bmatrix} \end{bmatrix}$

$V_{net.s.sl} = 0 \cdot kip$

Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$chk_{S7.18} := \text{if}(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, \text{"Slide Plates Req'd"}, \text{"OK"})$

$chk_{S7.18} = \text{"OK"}$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \begin{cases} \max\left(2, ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) & \text{if } \frac{ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ \max\left[2, \left(ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] & \text{otherwise} \end{cases}$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34)\,ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3\,ft$ on center.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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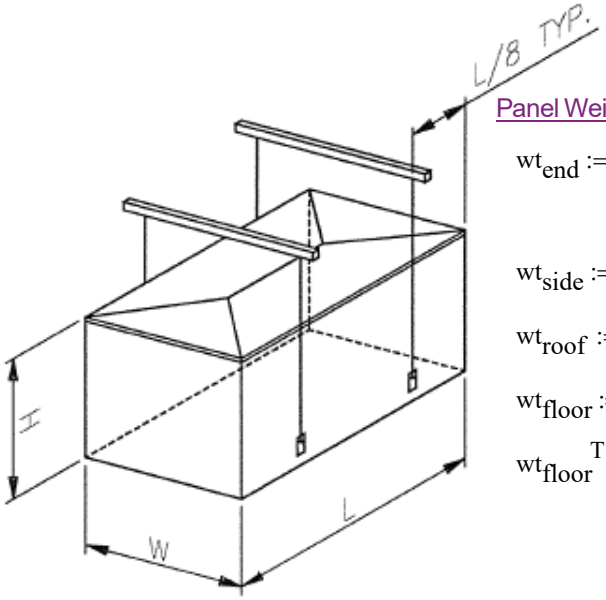
8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5153 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 442 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 679 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (438 \quad 437 \quad 436 \quad 435) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) $M/(0.9f_y d)$			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	30.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.222	0.199
12.00	32.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.257	0.226
12.00	34.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.295	0.253
12.00	36.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.336	0.280
12.00	30.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.210	0.154
12.00	32.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.244	0.175
12.00	34.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.280	0.196
12.00	36.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.318	0.216
12.00	30.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.204	0.135
12.00	32.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.236	0.153
12.00	34.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.271	0.171
12.00	36.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.309	0.190

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

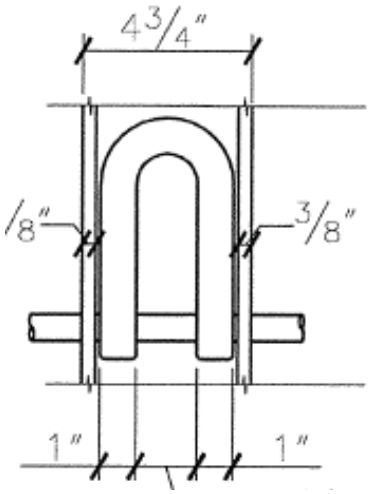
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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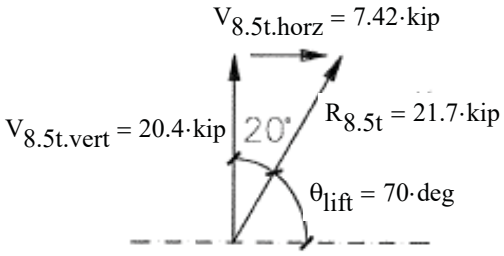
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

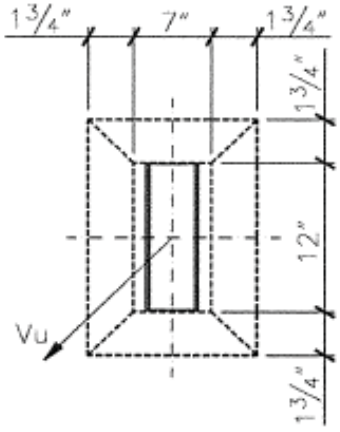
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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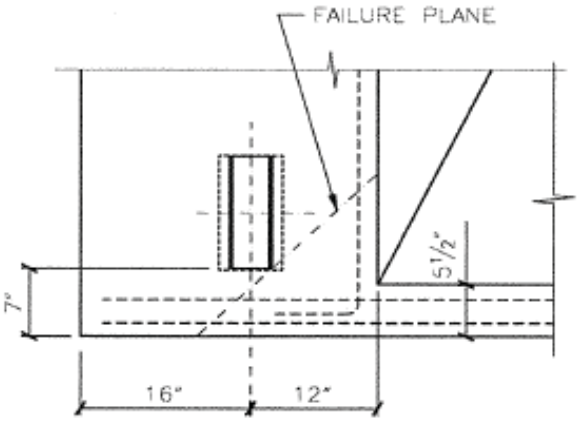
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

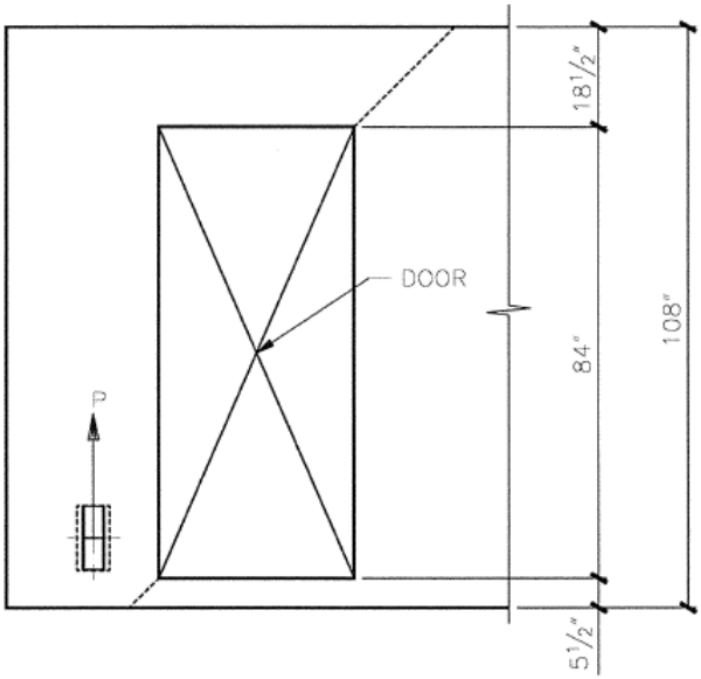
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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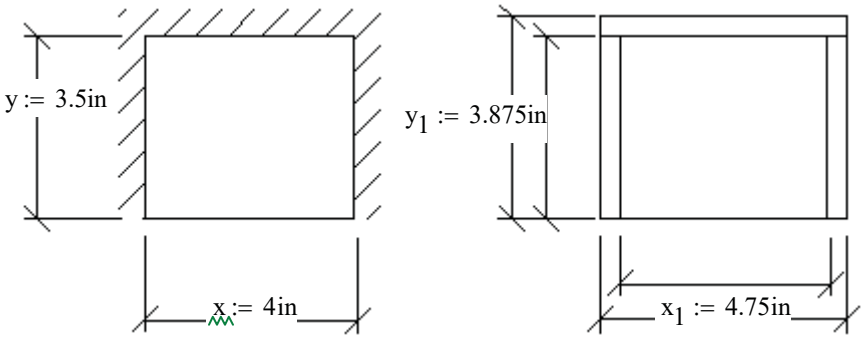
Bearing Pressure

Vertical



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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

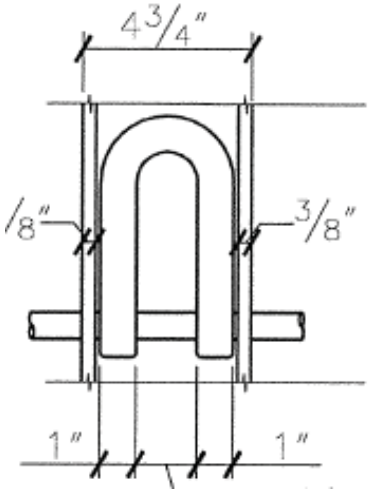
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

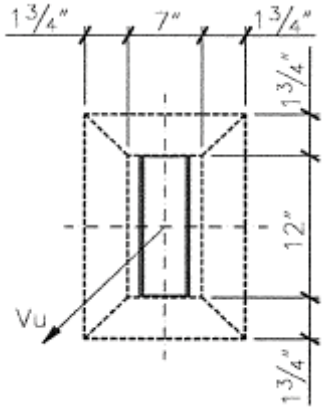
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

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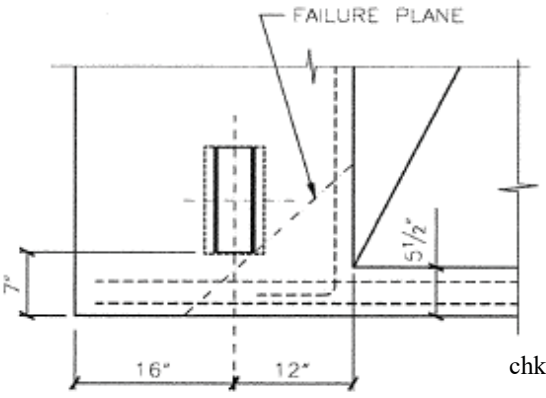
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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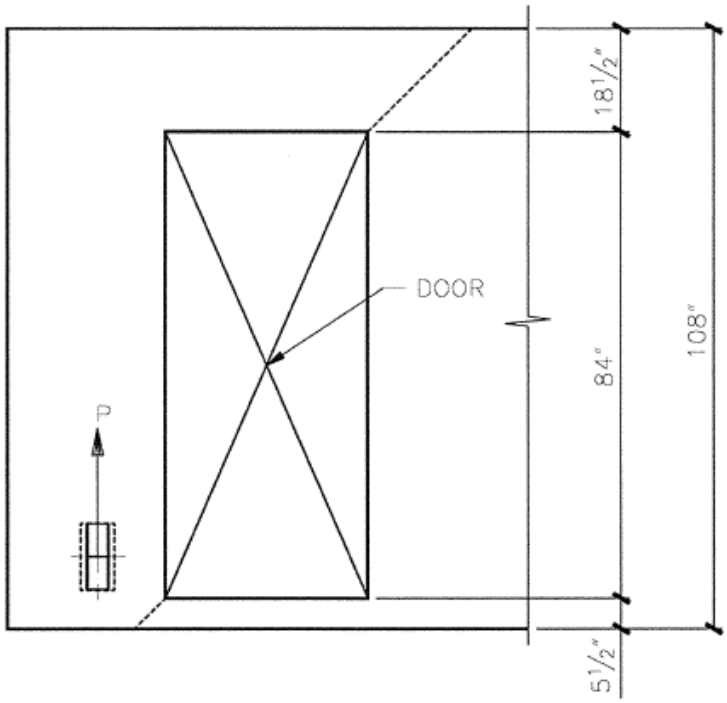


$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

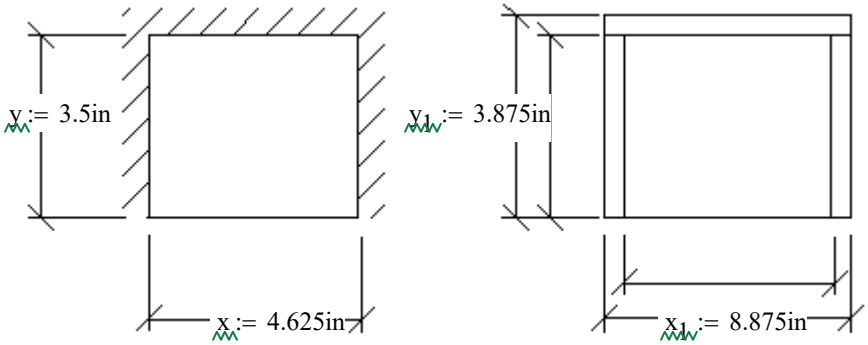
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

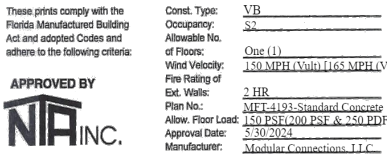
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 24.4 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 15.8 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2\max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 31.8 \cdot kip$ Wall Dead Load

$w_{ew} := 2\max(wt_{end})$ $w_{ew} = 10.3 \cdot kip$ Weight of End Wall

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Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 82.3 \cdot kip$ Factored Dead Load of Shelter

$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}}$ $CG = 7.1\,ft$ Center of gravity from bottom of shelter

Tie Down Bracket $Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 149.8 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}}$ $F_{turn} = 35.1 \cdot kip$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 260 \cdot kip \cdot ft$ Overtruning Moment

$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2}$ $M_{OT.Resist} = 314.8 \cdot kip \cdot ft$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -54.8 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}}$ $F_{OT.trans} = -6.4 \cdot kip$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans}\,DL$ $F_{net.slide.trans} = 18.6 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 9.3 \cdot kip$

Maximum Shear Force Per Tie-Down, θ_{min}

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 9.3 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$
$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right)$ $\phi P_{pl.FlrTie} = 26.928 \cdot kip$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.345$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.808$ < 1.0

$$\max\left(\frac{T_{tie.max}}{\phi P_{pl.FlrTie}}\right) + \frac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 1.154$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 1.154$ < 1.2

$chk_{8.Flr.Tie}$ = "OK"

9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 5 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 2.25 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 21.67 \quad U_{R3.25} = 0.046$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2) \cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 5 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 2.25 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 26.67 \quad U_{R4.75} = 0.037$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (29.292 \quad 31.292 \quad 33.292 \quad 35.292)\,ft$ Length of Floor

$wgt_{flr}^T = (37.559 \quad 37.469 \quad 37.39 \quad 37.32 \quad NaN \quad NaN \quad NaN \quad NaN \quad NaN) \cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 3.82 \cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 4.18 \cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.719 \\ 0.68 \\ 16.095 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 18.494 \quad U_F = 0.054$

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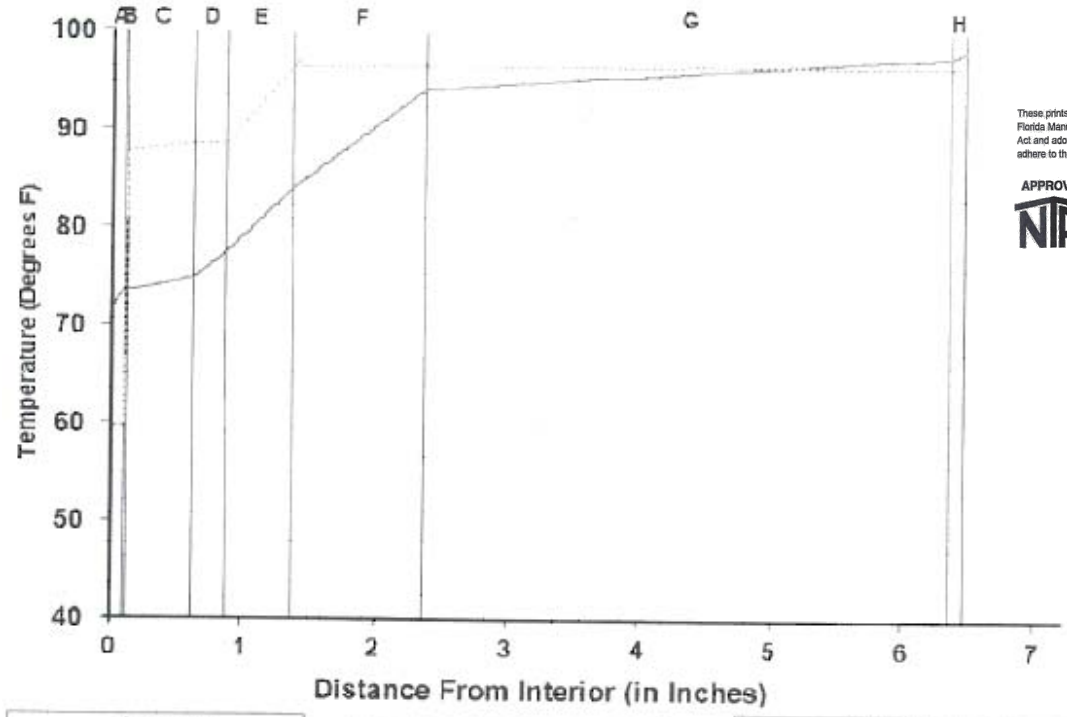
Dewpoint Analysis - Dow Chemical

Sam Cain



These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

May 17 2006 12:16:28 PM

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10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 65.753 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 935.2 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 1987 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$ Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.0101$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.242 \cdot in^2$ Req'd Area of Steel per foot Width

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Email: info@modularconnections.com

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 351 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG")$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 7.516\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 15.032\,ft$

Maximum allowable opening width

$w_{d.max} := \max(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4})$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

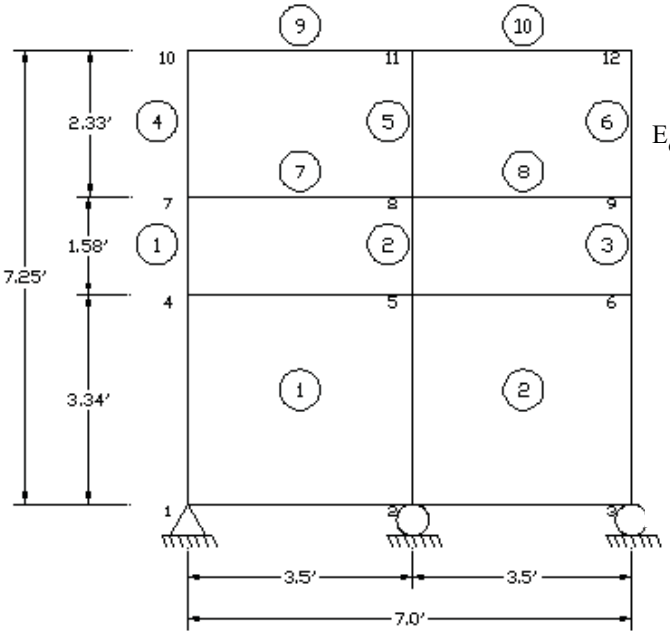
$chk_{opn.width} := if(w_d \geq w_{d.max}, "OK", "NG")$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.373 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.373 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.373 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2})$ In-Plane Force at Endwall



$V_u = 4.373 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in^2)	I (in^4)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$

Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$

Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.373 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.118 \cdot kip \cdot ft$

Shear

$\phi Vc_{6d.opn} := \phi_V \cdot 0.85 \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot \left(d_7 - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00111$

Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.14 \cdot in^2$

Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$

Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Email: info@modularconnections.com

APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Plan No.:	MET-4193-Standard Concrete
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTION, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

APPENDIX "B"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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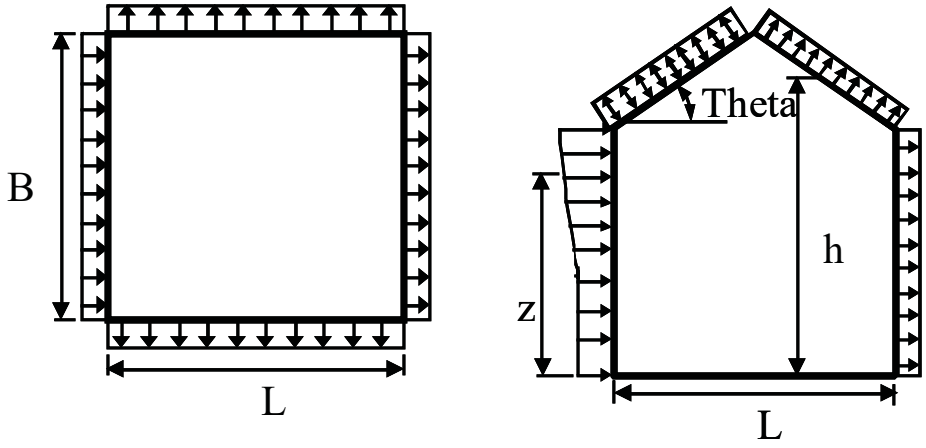
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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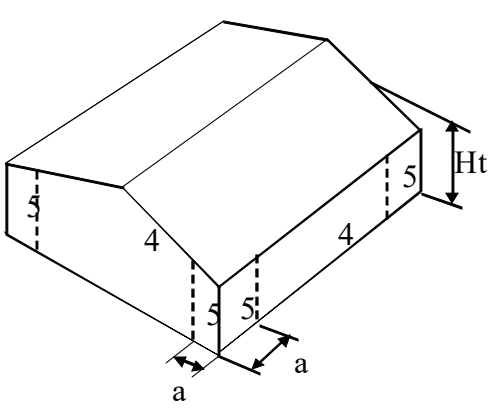
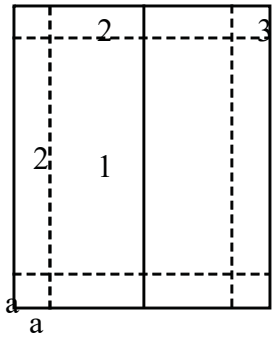
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

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Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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APPENDIX "C"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Occupancy: S2
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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

MODULAR CONNECTION, LLC.
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Email: info@modularconnections.com

MODULAR CONNECTIONS, LLC

Cover =

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

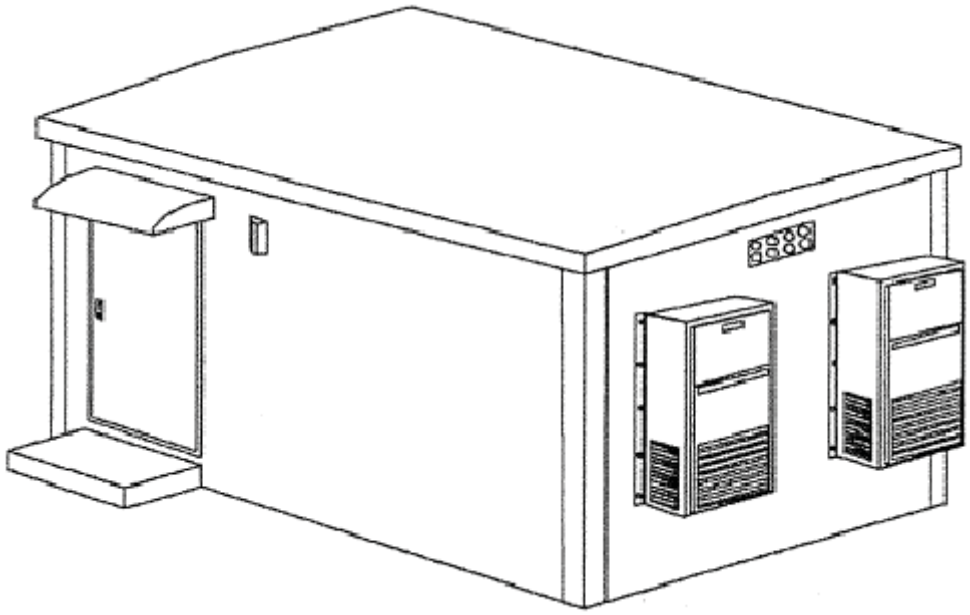
" IBC 2021 (ASCE 7-16) "

" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #3"

" REVISION - 0: DATE MARCH 11, 2024 "

""



" This analysis is the property of Modular Connections, LLC. It is not to be reproduced, "

" copied or traced in whole or part without our written consent. "

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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" WALL DESIGN ---------- S3.1 - S3.16 "
" FLOOR DESIGN ---------- S4.1 - S4.31 "
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" FIRE RATING & THERMAL EFFICIENCY---------- S9.1 - S9.3 "
" WALL OPENING DESIGN ---------- S10.1 - S10.4 "
""
""
" APPENDIX A - BULLET RESISTANCE TEST "
" APPENDIX B - AWINING / HOOD ATTACHMENT DESIGN "
" APPENDIX C - DEVELOPMENT OF REINFORCEMENT "

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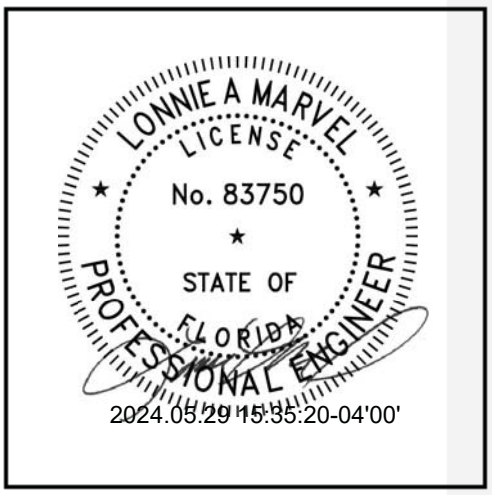
I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.



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Manufacturer:	Modular Connections, LLC

PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 30 ft - 36 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 37.4 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 46.7 psf"
"Roof Live Load ---------- 35 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 150 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 35.8 psf"
"Uplift Wind Pressure, MWFRS ---------- 53.5 psf"
"Zone 4 Wind Pressure, C & C ---------- 53.3 psf"
"Zone 5 Wind Pressure, C & C ---------- 65.8 psf"
"Zone 1 Wind Pressure, C & C ---------- 78.2 psf"
"Zone 2 Wind Pressure, C & C ---------- 103.2 psf"
"Zone 3 Wind Pressure, C & C ---------- 140.7 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 32 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 39.5 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.78"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.19"

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Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$) Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)

Welding electrodes : E70XX

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 35 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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Fire Rating of Ext. Walls: 2 HR
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot psf \qquad q_h = 41.616 \cdot psf \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 20.81 & 35.79 \\ -25.18 & -10.2 \\ -32.25 & -17.27 \\ -53.48 & -38.49 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 35.79 & 20.81 \\ -10.2 & -25.18 \\ -17.27 & -32.25 \\ -38.49 & -53.48 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 35.79 \\ 25.18 \\ 32.25 \\ -53.48 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 45.99 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 41.616 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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Manufacturer:	Modular Connections, LLC

(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -65.8 & -50.8 \\ 5 & 20 \\ -78.2 & -63.3 \\ 5 & 20 \\ -103.2 & -88.2 \\ 5 & 20 \\ -140.7 & -125.7 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -50.8 & -65.8 \\ 20 & 5 \\ -63.3 & -78.2 \\ 20 & 5 \\ -88.2 & -103.2 \\ 20 & 5 \\ -125.7 & -140.7 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 53.3 \\ 53.3 \\ 65.8 \\ 78.2 \\ 103.2 \\ 140.7 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 65.8 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 78.2 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 103.2 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 140.7 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 107.369 \cdot psf$$

Max Roof Uplift Pressure (average)

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 1.1 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1.06 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 1.166$ $S_{MS.ref} =$ "Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.777$ $S_{DS.ref} =$ "5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} =$ "Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} =$ "5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"

$$H_E = \begin{pmatrix} 9.479 \\ 10.479 \\ 11.146 \end{pmatrix} \text{ft} \qquad H_{E.ref} = \text{"Exterior Height of Building"}$$

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right] \qquad Ta = 0.104 \quad Ta_{ref} = \text{"Approx Per., Eqn (12.8-7)"}$$

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_0 = 0.216$ $T_s := \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_s = 1.079$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.1943$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E}\right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E}\right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1}\right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0342$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0\right)\right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max}\right), C_{s.min}\right)$ $C_s = 0.194$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

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2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 4 \cdot in$ Thickness @ ridge $t_{r.r} = 5.5 \cdot in$ $t_{r.a} = 4.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.1188 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 46.71 \cdot psf$ $DL_{r.mn.ref} :=$ "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 53.71 \cdot psf$ $DL_{r.mx.ref} :=$ "Max. Roof DL"

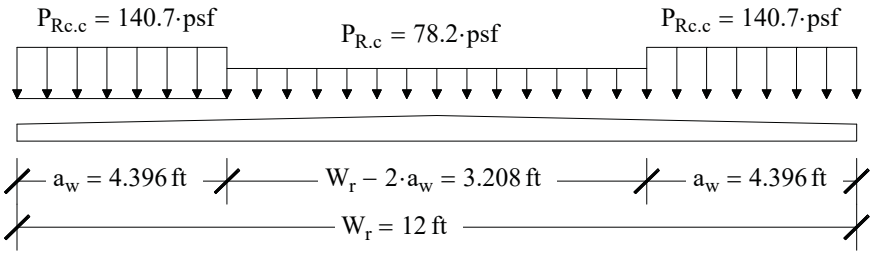
$w_{llr} = 35 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 140.7 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 78.2 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.396\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -24.137 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -111.745 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 8.35 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -8.35 \cdot psf$ Minimum Upward Roof Seismic Load

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Allowable No. of Floors: One (1)
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$LRFD = \begin{pmatrix} \text{"1a"} & \text{"1.4(D+F)"} \\ \text{"2a"} & \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"3a"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"3b+"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"3b-"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"4a+"} & \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"4a-"} & \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"5a+"} & \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"5a-"} & \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"6a+"} & \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"6a-"} & \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"7a+"} & \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"7a-"} & \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Wu_{S2.0} = \begin{pmatrix} 75.19 \\ 81.95 \\ 120.45 \\ 120.45 \\ 64.58 \\ 81.95 \\ 26.08 \\ 79.15 \\ 62.45 \\ 42.04 \\ -69.71 \\ 50.39 \\ 33.69 \end{pmatrix} \cdot psf$$

Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref} = \text{"Max Span of Roof Panel"}$

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 120.45 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -69.707 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

$Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$ $Vu_{ref} = \text{"Ultimate Shear"}$

Roof Live Load

$Mu_{2.1.mx} := |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r^2}{8}$ $Vu_{2.1.mx} := |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r}{2}$

$Mu_{2.1.mx} = 21.862\ kip \cdot in$ $Vu_{2.1.mx} = 0.662\ kip$

Option 1

$Mu_{2.1.mn} := \min\left(0kip \cdot in, |w_{u.mn}| \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$ $Vu_{2.1.mn} := \min\left(0kip, |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r}{2}\right)$

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$

$B_{2.1} = \text{"#4"}$ $B_{2.1.ref} = \text{"Transverse Rebar Size"}$ $S_{2.1} = 12 \cdot in$ $S_{2.1.ref} = \text{"Transverse Rebar Spacing"}$

$B_{2.2} = \text{"#4"}$ $B_{2.2.ref} = \text{"Longitudinal Rebar Size"}$ $S_{2.2} = 12 \cdot in$ $S_{2.2.ref} = \text{"Longitudinal Rebar Spacing"}$

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref} = \text{"Clear Cover, From Bot. w/ Keyway"}$

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref} = \text{"Clear Cover, From Bot. w/o Keyway"}$

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref} = \text{"Roof Width Bar Diam."}$ $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref} = \text{"Roof Width Bar Area."}$

$As_{2.1} = 0.2 \cdot in^2$ $As_{2.1.ref} = \text{"Roof Width Area Reinf."}$ $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref} = \text{"Roof Length Bar Diam."}$

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref} = \text{"Roof Length Bar Area."}$ $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref} = \text{"Roof Length Area Reinf."}$

$A_{s.mim.2} = 0.1188 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 3 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.2 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.0056$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.261 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 34.4 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.317 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.0254$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 31 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 21.9 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 1.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 2.908 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 10.264 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 2.908 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 2.181 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 0.662 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{F.2.3} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.5} = 0.261 \cdot in$ a_{ref} = "Depth of Comp. Block, Eqn (22.2.2.4.1)"

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $M_{n.2.3} = 19.4 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $c_{2.3} = 0.317 \cdot in$ c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\varepsilon_{2.3} = 0.01357$ ε_{ref} = "Tensile Strain, Extreme Layer"

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ ϕ_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\phi M_{n.2.3} = 17.5 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $Mu_{n.ref}$ = "Ultimate Moment, -ve"

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.5}$ = "OK"

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{V.2.4} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\lambda_{s.2.4} = 1$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.a} = 2.031 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $Vc_{2.4} = 2.031 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\phi Vc_{2.4} = 1.523 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Vu_{2.1.mn} = 0 \cdot kip$ Vu_{ref} = "Ultimate Shear"

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.6}$ = "OK"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 107.172 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 2.375 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 19.3 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 16.101 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 9.748 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 53.586 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -4.389 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 26.793 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 26.79 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 107.2 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 51.4 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.05519 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.19003 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.13484 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.435\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.091\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.526\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 250.7$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 35 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 35 \cdot psf$

$P_{W.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_0} =$ " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_1} =$ " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 65.8 \cdot psf$ $GC_{p.cc.max.ref_2} =$ "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 53.27 \\ 65.75 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 35 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.02 \\ 0.98 \\ 1.21 \\ 1.21 \\ 1.53 \\ 0.98 \\ 1.3 \\ 0.96 \\ 0.86 \\ 0.57 \\ 1.22 \\ 0.62 \\ 0.52 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 26.6 \\ 26.6 \\ 53.3 \\ 26.6 \\ 8.8 \\ -8.8 \\ 53.3 \\ 53.3 \\ 8.8 \\ -8.8 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 1.53 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.54 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.52 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.09 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.47 \cdot kip$

$P_{u.3b.h1}{}^T = (1.022 \;\; 0.981 \;\; 1.212 \;\; 1.212 \;\; 1.534 \;\; 0.981 \;\; 1.303 \;\; 0.965 \;\; 0.864 \;\; 0.571 \;\; 1.215 \;\; 0.621 \;\; 0.521) \cdot kip$

$M_{u.3b.h1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.27 \;\; 0.27 \;\; 0.539 \;\; 0.27 \;\; 0.089 \;\; -0.089 \;\; 0.539 \;\; 0.539 \;\; 0.089 \;\; -0.089) \cdot kip \cdot ft$

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
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$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.12 \;\; 0.12 \;\; 0.24 \;\; 0.12 \;\; 0.04 \;\; -0.04 \;\; 0.24 \;\; 0.24 \;\; 0.04 \;\; -0.04)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10ft$ & Roof Live Load $w_{llr} = 35\cdot psf$

$Pu_{3b.h2}^{T} = (1.09 \;\; 1.04 \;\; 1.27 \;\; 1.27 \;\; 1.59 \;\; 1.04 \;\; 1.36 \;\; 1.02 \;\; 0.92 \;\; 0.61 \;\; 1.25 \;\; 0.66 \;\; 0.56)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 8.8 \;\; -8.8 \;\; 53.3 \;\; 53.3 \;\; 8.8 \;\; -8.8)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2}) \quad P_{u.3b.h2.mx} = 1.59\cdot kip \qquad M_{u.3b.h2.mx} := \max(M_{u.3b.h2}) \quad M_{u.3b.h2.mx} = 0.67\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2}) \quad P_{u.3b.h2.mn} = 0.56\cdot kip \qquad M_{u.3b.h2.mn} := \min(M_{u.3b.h2}) \quad M_{u.3b.h2.mn} = -0.11\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|) \quad V_{u.3b.h2.mx} = 0.52\cdot kip$

$P_{u.3b.h2}^{T} = (1.086 \;\; 1.036 \;\; 1.267 \;\; 1.267 \;\; 1.589 \;\; 1.036 \;\; 1.358 \;\; 1.019 \;\; 0.919 \;\; 0.606 \;\; 1.25 \;\; 0.656 \;\; 0.556)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.333 \;\; 0.333 \;\; 0.666 \;\; 0.333 \;\; 0.11 \;\; -0.11 \;\; 0.666 \;\; 0.666 \;\; 0.11 \;\; -0.11)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.133 \;\; 0.133 \;\; 0.266 \;\; 0.133 \;\; 0.044 \;\; -0.044 \;\; 0.266 \;\; 0.266 \;\; 0.044 \;\; -0.044)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667ft$ & Roof Live Load $w_{llr} = 35\cdot psf$

$Pu_{3b.h3}^{T} = (1.13 \;\; 1.07 \;\; 1.3 \;\; 1.3 \;\; 1.63 \;\; 1.07 \;\; 1.39 \;\; 1.06 \;\; 0.95 \;\; 0.63 \;\; 1.27 \;\; 0.68 \;\; 0.58)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 8.8 \;\; -8.8 \;\; 53.3 \;\; 53.3 \;\; 8.8 \;\; -8.8)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3}) \quad P_{u.3b.h3.mx} = 1.63\cdot kip \qquad M_{u.3b.h3.mx} := \max(M_{u.3b.h3}) \quad M_{u.3b.h3.mx} = 0.76\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3}) \quad P_{u.3b.h3.mn} = 0.58\cdot kip \qquad M_{u.3b.h3.mn} := \min(M_{u.3b.h3}) \quad M_{u.3b.h3.mn} = -0.13\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|) \quad V_{u.3b.h3.mx} = 0.56\cdot kip$

$P_{u.3b.h3}^{T} = (1.128 \;\; 1.072 \;\; 1.303 \;\; 1.303 \;\; 1.625 \;\; 1.072 \;\; 1.394 \;\; 1.055 \;\; 0.955 \;\; 0.63 \;\; 1.274 \;\; 0.68 \;\; 0.58)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.379 \;\; 0.379 \;\; 0.758 \;\; 0.379 \;\; 0.125 \;\; -0.125 \;\; 0.758 \;\; 0.758 \;\; 0.125 \;\; -0.125)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.142 \;\; 0.142 \;\; 0.284 \;\; 0.142 \;\; 0.047 \;\; -0.047 \;\; 0.284 \;\; 0.284 \;\; 0.047 \;\; -0.047)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\quad S_{3.0}$ = "N/A" $\cdot in \qquad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \qquad d_{3b} := \frac{t_w}{2} - 0.25in \qquad d_{3b} = 1.75\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.558 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.056$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.947$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

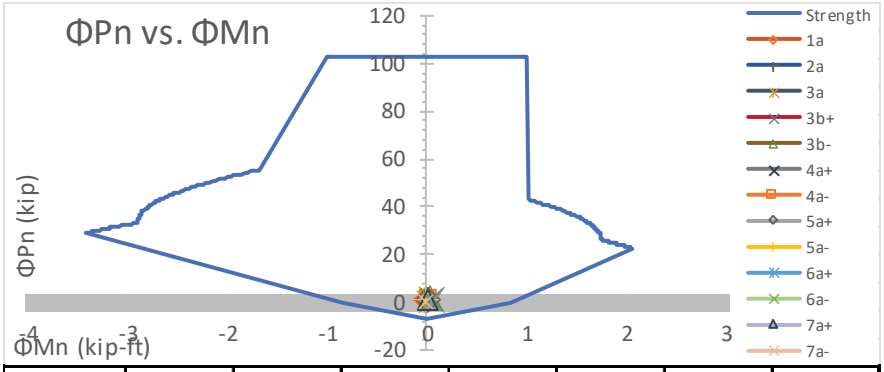
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.073$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.932$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.086$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.921$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.921$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.1%	1.0%	1.3%	2.95%	2.9%	6.0%	2.9%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.0%	1.0%	6.2%	5.9%	1.0%	1.0%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref}$ = "Min. Space to Adjacent Opening"

$b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref}$ = "Strip Width Btwn Openings"

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1}$ = "#6" $B_{3.1.ref}$ = "Wall Vert. Perim. Reinf." $B_{3.2}$ = "#4" $B_{3.2.ref}$ = "Wall Top. Perim. Reinf."

$N_{B.3.1} = 1$ $N_{B.3.1.ref}$ = "# Wall Vert. Perim. Bars" $N_{B.3.2} = 2$ $N_{B.3.2.ref}$ = "# Wall Top Perim. Bars"

$D_{3c.1} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.2} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.1} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.2} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.3}$ = "#6" $B_{3.3.ref}$ = "EW Bot Perim. Reinf." $B_{3.4}$ = "#4" $B_{3.4.ref}$ = "SW Bot Perim. Reinf."

$N_{B.3.3} = 1$ $N_{B.3.3.ref}$ = "# EW Bot Perim. Bars" $N_{B.3.4} = 2$ $N_{B.3.4.ref}$ = "# SW Bot Perim. Bars"

$D_{3c.3} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.4} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.3} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.4} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.5}$ = "#5" $B_{3.5.ref}$ = "Wall Opening Reinf." $D_{3c.5} = 0.625 \cdot in$ D_{ref} = "Bar Diam."

$N_{B.3.5} = 2$ $N_{B.3.5.ref}$ = "# Wall Opening Bars" $A_{3c.5} = 0.62 \cdot in^2$ A_{ref} = "Area of Reinf."

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 35 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 35.2 \;\; -35.2 \;\; 213.1 \;\; 213.1 \;\; 35.2 \;\; -35.2) \cdot plf$

$P_{u.3c.h1d1}{}^T = (4.09 \;\; 3.93 \;\; 4.85 \;\; 4.85 \;\; 6.14 \;\; 3.93 \;\; 5.21 \;\; 3.86 \;\; 3.46 \;\; 2.28 \;\; 4.86 \;\; 2.48 \;\; 2.08) \cdot kip$

$M_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.22 \;\; -0.22 \;\; 1.31 \;\; 1.31 \;\; 0.22 \;\; -0.22) \cdot kip \cdot ft$

$V_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.12 \;\; -0.12 \;\; 0.75 \;\; 0.75 \;\; 0.12 \;\; -0.12) \cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d1.mx} = 6.14 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.31 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.47 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.08 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.22 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 35 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

$W_{u.3c.h1d2}^{T} = (0\ \ 0\ \ 0\ \ 79.9\ \ 79.9\ \ 159.8\ \ 79.9\ \ 26.4\ \ -26.4\ \ 159.8\ \ 159.8\ \ 26.4\ \ -26.4)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.07\ \ 2.94\ \ 3.64\ \ 3.64\ \ 4.6\ \ 2.94\ \ 3.91\ \ 2.89\ \ 2.59\ \ 1.71\ \ 3.65\ \ 1.86\ \ 1.56)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0\ \ 0\ \ 0\ \ 0.49\ \ 0.49\ \ 0.98\ \ 0.49\ \ 0.16\ \ -0.16\ \ 0.98\ \ 0.98\ \ 0.16\ \ -0.16)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0\ \ 0\ \ 0\ \ 0.28\ \ 0.28\ \ 0.56\ \ 0.28\ \ 0.09\ \ -0.09\ \ 0.56\ \ 0.56\ \ 0.09\ \ -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 4.6\cdot kip$ $M_{u.3c.h1d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.1\cdot kip$

$P_{u.3c.h1d2.mn} = 1.56\cdot kip$ $M_{u.3c.h1d2.mn} = -0.16\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0\ \ 0\ \ 0\ \ 66.6\ \ 66.6\ \ 133.2\ \ 66.6\ \ 22\ \ -22\ \ 133.2\ \ 133.2\ \ 22\ \ -22)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.56\ \ 2.45\ \ 3.03\ \ 3.03\ \ 3.84\ \ 2.45\ \ 3.26\ \ 2.41\ \ 2.16\ \ 1.43\ \ 3.04\ \ 1.55\ \ 1.3)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0\ \ 0\ \ 0\ \ 0.41\ \ 0.41\ \ 0.82\ \ 0.41\ \ 0.13\ \ -0.13\ \ 0.82\ \ 0.82\ \ 0.13\ \ -0.13)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0\ \ 0\ \ 0\ \ 0.23\ \ 0.23\ \ 0.47\ \ 0.23\ \ 0.08\ \ -0.08\ \ 0.47\ \ 0.47\ \ 0.08\ \ -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 3.84\cdot kip$ $M_{u.3c.h1d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 0.92\cdot kip$

$P_{u.3c.h1d3.mn} = 1.3\cdot kip$ $M_{u.3c.h1d3.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0\ \ 0\ \ 0\ \ 106.5\ \ 106.5\ \ 213.1\ \ 106.5\ \ 35.2\ \ -35.2\ \ 213.1\ \ 213.1\ \ 35.2\ \ -35.2)\cdot plf$

$P_{u.3c.h2d1}^{T} = (4.34\ \ 4.14\ \ 5.07\ \ 5.07\ \ 6.36\ \ 4.14\ \ 5.43\ \ 4.08\ \ 3.67\ \ 2.42\ \ 5\ \ 2.63\ \ 2.22)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0\ \ 0\ \ 0\ \ 0.65\ \ 0.65\ \ 1.31\ \ 0.65\ \ 0.22\ \ -0.22\ \ 1.31\ \ 1.31\ \ 0.22\ \ -0.22)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0\ \ 0\ \ 0\ \ 0.37\ \ 0.37\ \ 0.75\ \ 0.37\ \ 0.12\ \ -0.12\ \ 0.75\ \ 0.75\ \ 0.12\ \ -0.12)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 6.36\cdot kip$ $M_{u.3c.h2d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.47\cdot kip$

$P_{u.3c.h2d1.mn} = 2.22\cdot kip$ $M_{u.3c.h2d1.mn} = -0.22\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0\ \ 0\ \ 0\ \ 79.9\ \ 79.9\ \ 159.8\ \ 79.9\ \ 26.4\ \ -26.4\ \ 159.8\ \ 159.8\ \ 26.4\ \ -26.4)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.26 \;\; 3.11 \;\; 3.8 \;\; 3.8 \;\; 4.77 \;\; 3.11 \;\; 4.07 \;\; 3.06 \;\; 2.76 \;\; 1.82 \;\; 3.75 \;\; 1.97 \;\; 1.67)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.16 \;\; -0.16 \;\; 0.98 \;\; 0.98 \;\; 0.16 \;\; -0.16)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.09 \;\; -0.09 \;\; 0.56 \;\; 0.56 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 4.77\cdot kip$ $M_{u.3c.h2d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.1\cdot kip$

$P_{u.3c.h2d2.mn} = 1.67\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.16\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 22 \;\; -22 \;\; 133.2 \;\; 133.2 \;\; 22 \;\; -22)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (2.71 \;\; 2.59 \;\; 3.17 \;\; 3.17 \;\; 3.97 \;\; 2.59 \;\; 3.39 \;\; 2.55 \;\; 2.3 \;\; 1.52 \;\; 3.13 \;\; 1.64 \;\; 1.39)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.13 \;\; -0.13 \;\; 0.82 \;\; 0.82 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.08 \;\; -0.08 \;\; 0.47 \;\; 0.47 \;\; 0.08 \;\; -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 3.97\cdot kip$ $M_{u.3c.h2d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 0.92\cdot kip$

$P_{u.3c.h2d3.mn} = 1.39\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 35.2 \;\; -35.2 \;\; 213.1 \;\; 213.1 \;\; 35.2 \;\; -35.2)\cdot plf$

$P_{u.3c.h3d1}^{T} = (4.51 \;\; 4.29 \;\; 5.21 \;\; 5.21 \;\; 6.5 \;\; 4.29 \;\; 5.58 \;\; 4.22 \;\; 3.82 \;\; 2.52 \;\; 5.1 \;\; 2.72 \;\; 2.32)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.22 \;\; -0.22 \;\; 1.31 \;\; 1.31 \;\; 0.22 \;\; -0.22)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.12 \;\; -0.12 \;\; 0.75 \;\; 0.75 \;\; 0.12 \;\; -0.12)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 6.5\cdot kip$ $M_{u.3c.h3d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.47\cdot kip$

$P_{u.3c.h3d1.mn} = 2.32\cdot kip$ $M_{u.3c.h3d1.mn} = -0.22\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 26.4 \;\; -26.4 \;\; 159.8 \;\; 159.8 \;\; 26.4 \;\; -26.4)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.38 \;\; 3.22 \;\; 3.91 \;\; 3.91 \;\; 4.88 \;\; 3.22 \;\; 4.18 \;\; 3.17 \;\; 2.86 \;\; 1.89 \;\; 3.82 \;\; 2.04 \;\; 1.74)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.16 \;\; -0.16 \;\; 0.98 \;\; 0.98 \;\; 0.16 \;\; -0.16)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.09 \;\; -0.09 \;\; 0.56 \;\; 0.56 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 4.88\cdot kip$ $M_{u.3c.h3d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.1\cdot kip$

$P_{u.3c.h3d2.mn} = 1.74\cdot kip$ $M_{u.3c.h3d2.mn} = -0.16\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 22 \;\; -22 \;\; 133.2 \;\; 133.2 \;\; 22 \;\; -22)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.13 \;\; -0.13 \;\; 0.82 \;\; 0.82 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.08 \;\; -0.08 \;\; 0.47 \;\; 0.47 \;\; 0.08 \;\; -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 4.06\cdot kip$ $M_{u.3c.h3d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 0.92\cdot kip$

$P_{u.3c.h3d3.mn} = 1.45\cdot kip$ $M_{u.3c.h3d3.mn} = -0.13\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - (0.75in + 0.5D_{3c.5})\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.465\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.099\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 0.916\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{(k_{wall}\cdot H_{wall})^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

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$$\delta_{3c.h1d1} := \frac{1}{1 - \frac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.271 \qquad \Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d1} = 0.787$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

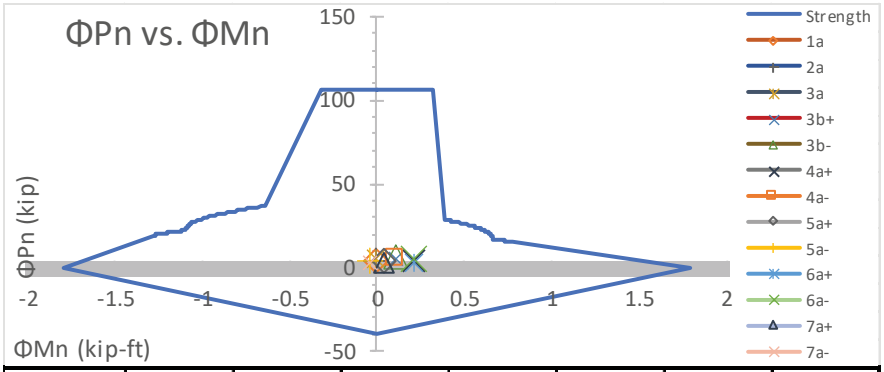
$$\delta_{3c.h2d1} := \frac{1}{1 - \frac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d1} = 1.374 \qquad \Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1} \qquad \Omega_{\delta.3c.h2d1} = 0.728$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h3d1} := \frac{1}{1 - \frac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d1} = 1.906 \qquad \Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1} \qquad \Omega_{\delta.3c.h3d1} = 0.525$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\Omega_{\delta.3c.d1} := \min\left(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1}\right) \qquad \Omega_{\delta.3c.d1} = 0.525$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.2%	4.0%	4.9%	7.43%	7.9%	14.2%	7.6%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.0%	3.6%	13.2%	14.8%	2.6%	2.2%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h1d2} := \frac{1}{1 - \frac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.271 \qquad \Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d2} = 0.787$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h2d2} := \frac{1}{1 - \frac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d2} = 1.257 \qquad \Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1} \qquad \Omega_{\delta.3c.h2d2} = 0.796$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h3d2} := \frac{1}{1 - \frac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d2} = 1.554 \qquad \Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1} \qquad \Omega_{\delta.3c.h3d2} = 0.643$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\Omega_{\delta.3c.d2} := \min\left(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2}\right) \qquad \Omega_{\delta.3c.d2} = 0.643$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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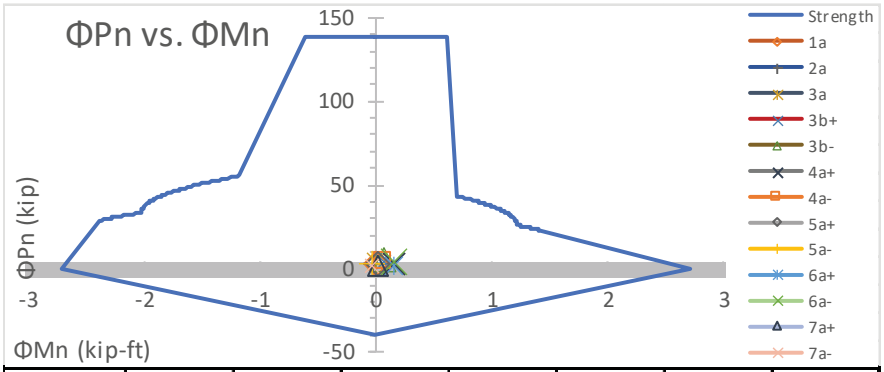
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.4%	2.3%	2.8%	3.21%	3.5%	6.3%	3.2%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.3%	2.1%	6.1%	6.4%	1.5%	1.3%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.154 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.867$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

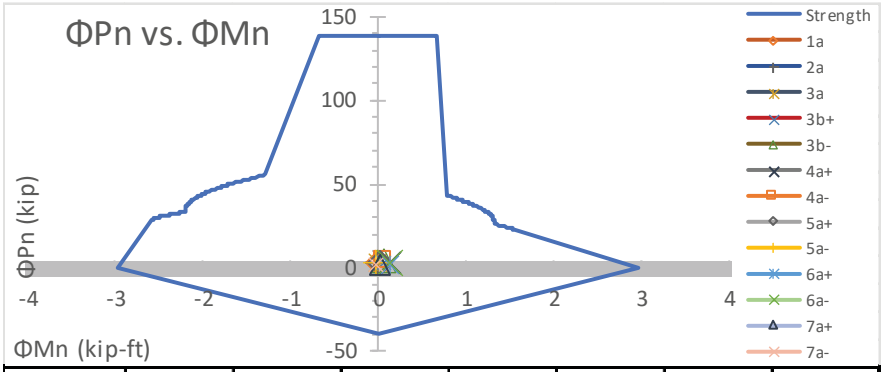
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.205 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.83$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.423 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.703$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.703$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.0%	1.9%	2.4%	2.41%	2.9%	4.8%	2.5%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.9%	1.7%	4.6%	4.8%	1.2%	1.0%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

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d. Wall - Header Check (Above Opening) $w_{llr} = 35 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in \quad \text{Min Header Height for Opening Widths of} \quad w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$$

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$$Wu_{3d.0}^{T} = (662.7 \quad 673 \quad 904 \quad 904 \quad 1226.1 \quad 673 \quad 995.1 \quad 656.2 \quad 556 \quad 370.2 \quad 1014.4 \quad 420.3 \quad 320.1) \cdot plf$$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1226.1 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \qquad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in \qquad A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for for:
$w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min\left(Vc_{3d.0.a}, Vc_{3d.0.b}\right)$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot \left(w_{open_0}\right)^2}{8}$ $Mu_{3d.0} = 5.52 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max\left(\left|wu_{3d.0mx}\right|, \left|wu_{3d.0mn}\right|\right) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 3.68 \cdot kip$

$chk_{3d.1} := \text{if}\left(\phi Mn_{3d.0} \geq Mu_{3d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot \left[f_y \cdot \left(0.5 \cdot h_{3d_0}\right)\right]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := \text{if}\left(chk_{3d.2} = \text{"OK"}, \text{"OK"}, \text{if}\left(chk_{3d.2} = \text{"Stirrups Req'd"}, \text{if}\left(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, \text{"NG"}, \text{"OK"}\right), \text{"OK"}\right)\right)$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (562.2 \;\; 586.9 \;\; 817.9 \;\; 817.9 \;\; 1140 \;\; 586.9 \;\; 909 \;\; 570.1 \;\; 469.9 \;\; 314.2 \;\; 958.4 \;\; 364.3 \;\; 264.1) \cdot plf$

$wu_{3d.1mx} := \max\left(Wu_{3d.1}\right)$ $wu_{3d.1mx} = 1140 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min\left(Wu_{3d.1}, 0plf\right)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}\left(a_{3d.1.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots$$
$$+ A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := \text{root}\left(f_{3d.1}\left(a_{3d.1.g}\right), a_{3d.1.g}\right)$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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$F2_{3d.1} := A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right]$ $F2_{3d.1} = 11.7 \cdot kip$

$F3_{3d.1} := A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right]$ $F3_{3d.1} = 7.2 \cdot kip$

$F4_{3d.1} := A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$ $F4_{3d.1} = 18.6 \cdot kip$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772 \cdot kip$

$Mn_{3d.1} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} \cdot \frac{a_{3d.1}}{2} + F1_{3d.1} \cdot d_{3d.1_0} + F2_{3d.1} \cdot d_{3d.1_1} \ldots + F3_{3d.1} \cdot d_{3d.1_2} + F4_{3d.1} \cdot d_{3d.1_3}$ $Mn_{3d.1} = 38.5 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}} \cdot d_{3d.1_3}$ $\varepsilon_{3d.1} = 0.014$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{3d.1} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.1} := \phi_{3d.1} \cdot Mn_{3d.1}$ $\phi Mn_{3d.1} = 34.64 \cdot k \cdot ft$ $Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375 \cdot in$

$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) \cdot t_w}$ $\rho_{3d.1} = 0.0062$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.1} := \sum \begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.1_0}\right)}\right]$ $\rho'_{3d.1} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10 \cdot in}}}\right)$ $\lambda_{s.3d.1} = 0.876$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.1.a} := 8 \cdot \lambda_{s.3d.1} \cdot \lambda \cdot \rho_{3d.1}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.a} = 5.48 \cdot kip$

$Vc_{3d.1.b} = 21.31 \cdot kip$

$\phi Vc_{3d.1} := \phi_V \cdot \min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right)$ $\phi Vc_{3d.1} = 4.1 \cdot kip$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.1} := \frac{wu_{3d.1mx} \cdot \left(w_{open_1}\right)^2}{8}$ $Mu_{3d.1} = 2.28 \cdot kip \cdot ft$ $Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right) \cdot w_{open_1}}{2}$ $Vu_{3d.1} = 2.28 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h_{3d_1}\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (541.1 \quad 568.8 \quad 799.8 \quad 799.8 \quad 1121.9 \quad 568.8 \quad 890.9 \quad 552 \quad 451.8 \quad 302.4 \quad 946.6 \quad 352.5 \quad 252.3) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1121.9 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_2} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_2} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 2.28 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 1.68 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2} \\ \text{"Min. Stirrups Req'd"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2} \\ \text{"Stirrups Req'd"} & \text{otherwise} \end{cases}$ $\quad chk_{3d.10} = \text{"OK"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 3.68 \cdot psf$		$M_{E.4a} = 0.001 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.003 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.235 \\ 0.235 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.081 \\ 0.081 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.193 \\ 0.193 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.063 \\ 0.063 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.152 \\ 0.152 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 11.35 \cdot plf$		$M_{E.4b} = 0.124 \cdot kip \cdot ft$	$V_{E.4b} = 0.053 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.52 \\ 6.52 \\ 0.58 \\ 0.58 \\ 0.7 \\ 0.7 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.8 \\ 2.8 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.43 \\ 5.43 \\ 0.58 \\ 0.58 \\ 0.7 \\ 0.7 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.33 \\ 2.33 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.34 \\ 4.34 \\ 0.58 \\ 0.58 \\ 0.7 \\ 0.7 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.86 \\ 1.86 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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Allowable No. of Floors: One (1)
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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.co

$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (12 \quad 13 \quad 14 \quad 15)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (29.29 \quad 31.29 \quad 33.29 \quad 35.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (6.86 \quad 6.96 \quad 7.04 \quad 7.12) \cdot psf$ Floor Transverse Ribs

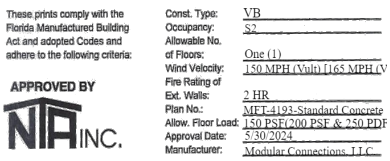
$wgt_{f.l.rib}^T = (3.63 \quad 3.63 \quad 3.64 \quad 3.64) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (3.27 \quad 3.06 \quad 2.88 \quad 2.72) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (4.13 \quad 4.15 \quad 4.16 \quad 4.17) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (37.6 \quad 37.5 \quad 37.4 \quad 37.3) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (41.6 \quad 41.5 \quad 41.4 \quad 41.3) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"



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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

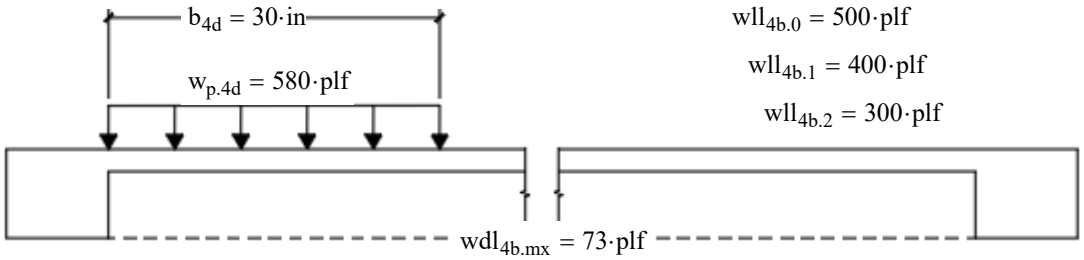
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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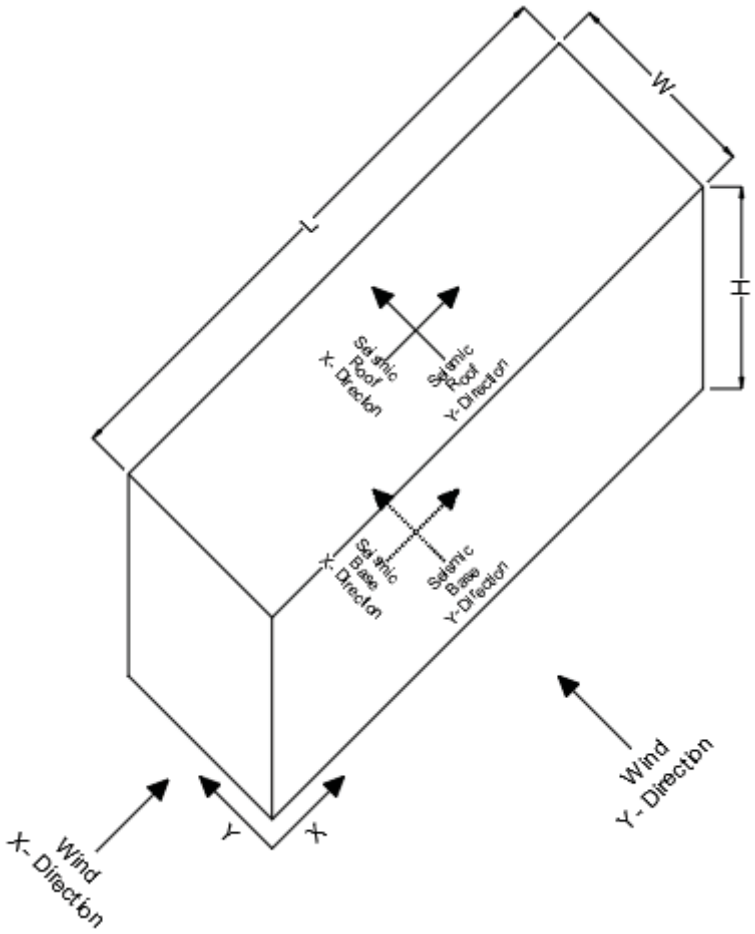
5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx}\cdot(L_r)\cdot(W_r)$

$Wt_{rf.mn} := DL_{r.mn}\cdot(L_r)\cdot(W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25\cdot w_{llr}, 0psf)$

$w_{llr.eq} = 8.8\cdot psf$

$Wt_{25ll} := w_{llr.eq}\cdot[(L_r)\cdot(W_r)]$

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$Wt_{rf.mx}^{T} = (19.5 \quad 20.8 \quad 22.1 \quad 23.4)\cdot kip$

Max Roof Weight, $w_{llr} = 35\cdot psf$

$Wt_{25ll}^{T} = (3.2 \quad 3.4 \quad 3.6 \quad 3.8)\cdot kip$

25% Roof LLr 0, $w_{llr} = 35\cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (23 \quad 24 \quad 26 \quad 27)\cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (17 \quad 18.1 \quad 19.2 \quad 20.4)\cdot kip$

Min Roof Weight, $w_{llr} = 35\cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75\cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3}\cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128)\cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (113.5 \quad 125.5 \quad 133.5)\cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 110.75 \\ 122.75 \\ 130.75 \end{pmatrix}\cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r}\cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r}\cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r}\cdot el_{avg.rf_2}$

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Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h1.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h2.mx}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{w.h0.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h1.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{w.h2.mn}^T = (29 \quad 30.4 \quad 31.8 \quad 33.2) \cdot kip$

$Wt_{EQ.w.h0}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h1}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

$Wt_{EQ.w.h2}^T = (33.4 \quad 35.1 \quad 36.7 \quad 38.3) \cdot kip$

Max Wall Weight, $H_{wall_0} = 9\,ft$

Max Wall Weight, $H_{wall_1} = 10\,ft$

Max Wall Weight, $H_{wall_2} = 10.7\,ft$

Min Wall Weight, $H_{wall_0} = 9\,ft$

Min Wall Weight, $H_{wall_1} = 10\,ft$

Min Wall Weight, $H_{wall_2} = 10.7\,ft$

Seismic Wall Weight, $H_{wall_0} = 9\,ft$

Seismic Wall Weight, $H_{wall_1} = 10\,ft$

Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

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$el_{top.w} := h_{wl\_fl} + H_{wall}$

$el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$

Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean\left(h_{wl\_fl}, el_{top.w_0}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_1}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_2}\right) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$

$W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$

$W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if\left(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf\right)$

$w_{llf.0.eq} = 62.5 \cdot psf$

$Wt_{25llf0} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.1.eq} := if\left(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf\right)$

$w_{llf.1.eq} = 50 \cdot psf$

$Wt_{25llf1} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.2.eq} := if\left(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf\right)$

$w_{llf.2.eq} = 37.5 \cdot psf$

$Wt_{25llf2} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (13.3 \quad 14.2 \quad 15.1 \quad 16) \cdot kip$

Max Floor Weight

$Wt_{f.mn}^T = (12.1 \quad 12.8 \quad 13.6 \quad 14.4) \cdot kip$

Min Floor Weight

$Wt_{25llf0}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (20.1 \quad 21.4 \quad 22.8 \quad 24.2) \cdot kip$

25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$

Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

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$Wt_{EQ.f.f0}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (33.4 \quad 35.7 \quad 37.9 \quad 40.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 45.99\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (12.8 \quad 13.6 \quad 14.5 \quad 15.4)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (14.1 \quad 15.1 \quad 16 \quad 17)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (15 \quad 16 \quad 17.1 \quad 18.1)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 5.09\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 5.62\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 5.98\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (6.4 \quad 6.8 \quad 7.3 \quad 7.7)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (7.1 \quad 7.5 \quad 8 \quad 8.5)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (7.5 \quad 8 \quad 8.5 \quad 9)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 2.54\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 2.81\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 2.99\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.194$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{EQ.bh0f0}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh1f0} := C_s \cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.bh2f0}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh2f0} := C_s \cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.bh0f1}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh0f1} := C_s \cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

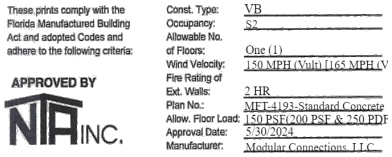
$Wt_{EQ.bh1f1}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh1f1} := C_s \cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$



$Wt_{EQ.bh2f1}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh2f1} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.bh0f2}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh0f2} := C_s \cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.bh1f2}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh1f2} := C_s \cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.bh2f2}^{T} = (90 \quad 95 \quad 100 \quad 106)\cdot kip$

$V_{EQ.bh2f2} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (17 \quad 18 \quad 19 \quad 21)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$Wt_{EQ.rh0}^{T} = (39 \quad 42 \quad 44 \quad 46)\cdot kip$

$V_{EQ.rh0} := C_s \cdot Wt_{EQ.rh0}$

Roof Diap. Wt/Shear

$w_{llr} = 35\cdot psf \quad H_{wall_0} = 9\,ft$

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$V_{EQ.rh0}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

$Wt_{EQ.rh1}^{T} = (39 \quad 42 \quad 44 \quad 46)\cdot kip$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

$Wt_{EQ.rh2}^{T} = (39 \quad 42 \quad 44 \quad 46)\cdot kip$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^{T} = (8 \quad 8 \quad 9 \quad 9)\cdot kip$

Roof Diap. Wt/Shear
$w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Roof Diap. Wt/Shear $w_{llr} = 35\cdot psf$
$H_{wall_2} = 10.7\,ft$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r}$ $w_{EQh0.y}^{T} = (253 \quad 251 \quad 249 \quad 248)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r}$ $w_{EQh1.y}^{T} = (253 \quad 251 \quad 249 \quad 248)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r}$ $w_{EQh2.y}^{T} = (253 \quad 251 \quad 249 \quad 248)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_2} = 10.67ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r}$ $w_{EQh0.x}^{T} = (639 \quad 676 \quad 714 \quad 751)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r}$ $w_{EQh1.x}^{T} = (639 \quad 676 \quad 714 \quad 751)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r}$ $w_{EQh2.x}^{T} = (639 \quad 676 \quad 714 \quad 751)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_2} = 10.67ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r}$ $w_{W.h0.y}^{T} = (210 \quad 211 \quad 211 \quad 212)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r}$ $w_{W.h1.y}^{T} = (233 \quad 233 \quad 234 \quad 234)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r}$ $w_{W.h2.y}^{T} = (247 \quad 248 \quad 248 \quad 249)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$

$w_{W.h0.x} = 211.9\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$

$w_{W.h1.x} = 234.3\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$

$w_{W.h2.x} = 249.2\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_2} = 10.7\,ft$

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$M_{u.bh0.y}^{T} = (29.1 \quad 32.8 \quad 36.8 \quad 40.9)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (29.1 \quad 32.8 \quad 36.8 \quad 40.9)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (29.1 \quad 32.8 \quad 36.8 \quad 40.9)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.y}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.bh0.x}^{T} = (11.5 \quad 12.2 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (11.5 \quad 12.2 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (11.5 \quad 12.2 \quad 12.8 \quad 13.5)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.x}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.b.h0.y}^{T} = (24.2 \quad 27.6 \quad 31.1 \quad 34.9)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (26.8 \quad 30.5 \quad 34.4 \quad 38.6)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (28.5 \quad 32.4 \quad 36.6 \quad 41.1)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$V_{u.b.h0.y}^{T} = (3.2 \quad 3.4 \quad 3.6 \quad 3.8)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (3.5 \quad 3.8 \quad 4 \quad 4.3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (3.8 \quad 4 \quad 4.3 \quad 4.5)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 3.8\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 4.2\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 4.5\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

$v_{u.b.h0.x} = 1.3\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.4\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = "\#6"$ $B_{7.1.ref} = "Roof\ Perim.\ Reinf."$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = "\#\ Roof\ Perim.\ Bars"$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = "Roof\ Perim.\ Reiinf.\ Bar\ Diam."$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = "Roof\ Perim.\ Reinf.\ Area"$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := "Roof\ Diaph.Reinf."$ $\rho_{b.1} := \frac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00111$

$d_{b.1} := W_r - 0.75in - \frac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \frac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 2.51\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \frac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\frac{d_{b.1}}{2} - \frac{a_{b.1}}{2}\right)$ $M_{b.1} = 381.7\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \frac{a_{b.1}}{0.85}$ $c_{b.1} = 3\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\frac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.14216$ Strain in Ten. Rebar

$\phi_{b.1} := if\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, if\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 343.6\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\frac{M_{u.bh0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.bh1.y}}{\phi M_{b.1_1}} \quad \frac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 8.5 & 9.5 & 10.7 & 11.9 \\ 8.5 & 9.5 & 10.7 & 11.9 \\ 8.5 & 9.5 & 10.7 & 11.9 \end{pmatrix}\cdot\%$

$chk_{b.1a} := if\left(max\left(Unity_{b.1a}\right) < 1, "OK", "NG"\right)$

$chk_{b.1a} = "OK"$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.1b}^T = \begin{pmatrix} 7 & 8 & 9.1 & 10.2 \\ 7.8 & 8.9 & 10 & 11.2 \\ 8.3 & 9.4 & 10.7 & 12 \end{pmatrix} \cdot \%$$

$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.64 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 2.51 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$

$d_{b.2}^T = (30.26 \quad 32.26 \quad 34.26 \quad 36.26)\,ft$

$\rho_{b.2}^T = (0.00044 \quad 0.00041 \quad 0.00039 \quad 0.00037)$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.1}}{2} \right)$$

$M_{b.2a}^T = (976 \quad 1041 \quad 1106 \quad 1171) \cdot kip \cdot ft$ Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 3 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$\varepsilon_{t.b.2a}^T = (0.3659 \quad 0.3903 \quad 0.4147 \quad 0.4391)$ Strain in Rebar

$\phi_{b.2} := if\left[min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50 \right] \right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^T = (879 \quad 937 \quad 995 \quad 1054) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.2a}^T = \begin{pmatrix} 1.3 & 1.3 & 1.3 & 1.3 \\ 1.3 & 1.3 & 1.3 & 1.3 \\ 1.3 & 1.3 & 1.3 & 1.3 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.2b}^T = \begin{pmatrix} 0.4 & 0.4 & 0.4 & 0.4 \\ 0.5 & 0.4 & 0.4 & 0.4 \\ 0.5 & 0.5 & 0.5 & 0.4 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 576 \cdot in^2$ Roof Shear Area for $t_{r.e} = 4 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00111$

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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e} \cdot L_{r_0} \\ t_{r.e} \cdot L_{r_1} \\ t_{r.e} \cdot L_{r_2} \\ t_{r.e} \cdot L_{r_3} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 1456 \\ 1552 \\ 1648 \\ 1744 \end{pmatrix} \cdot in^2$$

Roof Shear Area for $t_{r.e} = 4 \cdot in$ & $L_r = \begin{pmatrix} 4368 \\ 4656 \\ 4944 \\ 5232 \end{pmatrix} \cdot \frac{in^2}{ft}$

$\rho_{b.2}^{T} = (0.00044 \quad 0.00041 \quad 0.00039 \quad 0.00037)$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv} \cdot \left(2 \cdot \sqrt{f'_c} + \rho \cdot f_y\right)$ $V_{b.3a.1.mx} = 115.7 \cdot kip$

Max Allowable for $t_{r.e} = 4 \cdot in$ & $W_r = 12 \cdot ft$

$V_{b.3a.2.mx}^{T} = (233.7 \quad 246.6 \quad 259.5 \quad 272.4) \cdot kip$

Max Allowable Shear for $t_{r.e} = 4 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$V_{b.3b.1} = 32.8 \cdot kip$ Nominal Shear Strength for $t_{r.e} = 4 \cdot in$ & $W_r = 12 \cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

Roof Diaph. Shear Strength for $t_{r.e} = 4 \cdot in$ & $W_r = 12 \cdot ft$ $\phi V_{c.b.3a.1} = 24.6 \cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$V_{b.3b.2}^{T} = (83 \quad 88.5 \quad 94 \quad 99.4) \cdot kip$

Nominal Shear Strength for $t_{r.e} = 4 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$\phi V_{c.b.3a.2}^{T} = (62.3 \quad 66.4 \quad 70.5 \quad 74.6) \cdot kip$

Roof Diaph. Shear Strength for $t_{r.e} = 4 \cdot in$ & $L_r^{T} = (30.3 \quad 32.3 \quad 34.3 \quad 36.3) \cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 15.6 & 16.5 & 17.4 & 18.3 \\ 15.6 & 16.5 & 17.4 & 18.3 \\ 15.6 & 16.5 & 17.4 & 18.3 \end{pmatrix} \cdot \%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 13 & 13.8 & 14.7 & 15.6 \\ 14.3 & 15.3 & 16.3 & 17.3 \\ 15.2 & 16.3 & 17.3 & 18.4 \end{pmatrix} \cdot \%$$

$chk_{b.3d} = "OK"$

Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 35 \cdot psf$

$Unity_{b.3g}^{T} = (6.2 \quad 6.1 \quad 6.1 \quad 6) \cdot \%$

$chk_{b.3j} = "OK"$

Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\ ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 2 & 1.9 & 1.8 & 1.7 \\ 2.3 & 2.1 & 2 & 1.9 \\ 2.4 & 2.3 & 2.1 & 2 \end{pmatrix} \cdot \%$$

$chk_{b.3j} = "OK"$

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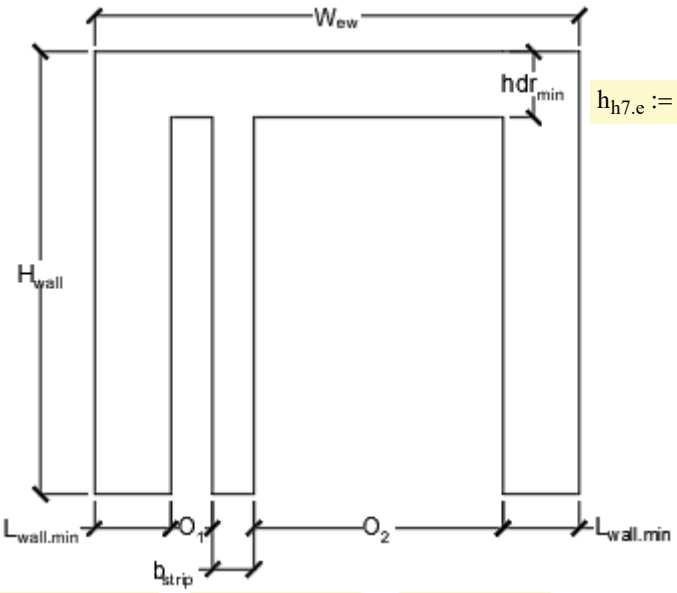
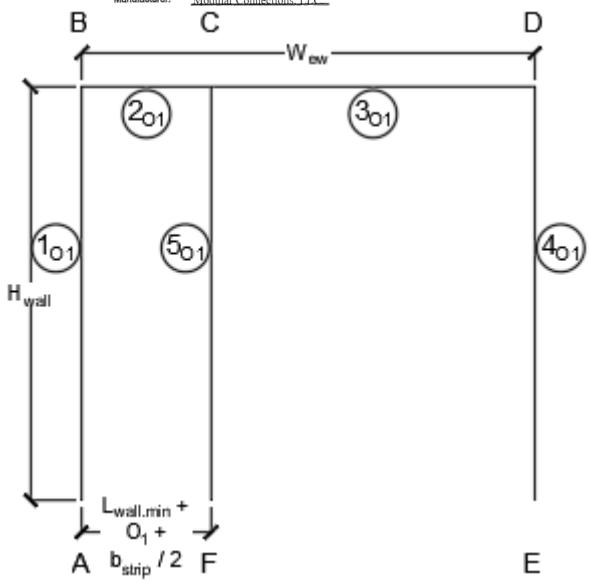
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

END WALL DESIGN



$h_{h7.e} := 19in$



$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right) \qquad Veq_{ewho}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5) \cdot kip \qquad \text{Height } H_{wall_0} = 9\,ft$$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right) \qquad Veq_{ewh1}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5) \cdot kip \qquad \text{Height } H_{wall_1} = 10\,ft$$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right) \qquad Veq_{ewh2}^{T} = (3.8 \quad 4.1 \quad 4.3 \quad 4.5) \cdot kip \qquad \text{Height } H_{wall_2} = 10.7\,ft$$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix} \qquad K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \dfrac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \dfrac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint C

$$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix} \qquad K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \dfrac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \dfrac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

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Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}{}^T = (6.5 \quad 6.8 \quad 7.2 \quad 7.6) \cdot kip \cdot ft$ $V_{L.ewh0}{}^T = (1.2 \quad 1.3 \quad 1.4 \quad 1.5) \cdot kip$

$M_{hdrh0}{}^T = (4.7 \quad 5 \quad 5.3 \quad 5.6) \cdot kip \cdot ft$ $V_{hdrh0}{}^T = (1.6 \quad 1.7 \quad 1.8 \quad 1.9) \cdot kip$

$M_{b.striph0}{}^T = (7.1 \quad 7.5 \quad 8 \quad 8.4) \cdot kip \cdot ft$ $V_{b.striph0}{}^T = (1.5 \quad 1.6 \quad 1.6 \quad 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10 ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

$DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}{}^T = (7.4 \quad 7.9 \quad 8.3 \quad 8.7) \cdot kip \cdot ft$ $V_{L.ewh1}{}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$M_{hdrh1}^T = (5.9 \quad 6.3 \quad 6.6 \quad 7)\cdot kip\cdot ft$

$V_{hdrh1}^T = (2.6 \quad 2.8 \quad 2.9 \quad 3.1)\cdot kip$

$M_{b.striph1}^T = (8 \quad 8.4 \quad 8.9 \quad 9.4)\cdot kip\cdot ft$

$V_{b.striph1}^T = (1.5 \quad 1.6 \quad 1.7 \quad 1.2)\cdot kip$

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4\cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix}\cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119\cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix}\cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4\cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh2}^T = (8 \quad 8.4 \quad 8.9 \quad 9.4)\cdot kip\cdot ft$

$V_{L.ewh2}^T = (1.4 \quad 1.4 \quad 1.5 \quad 1.6)\cdot kip$

$M_{hdrh2}^T = (6.4 \quad 6.8 \quad 7.2 \quad 7.6)\cdot kip\cdot ft$

$V_{hdrh2}^T = (2.9 \quad 3 \quad 3.2 \quad 3.4)\cdot kip$

$M_{b.striph2}^T = (8.5 \quad 9 \quad 9.5 \quad 10)\cdot kip\cdot ft$

$V_{b.striph2}^T = (1.5 \quad 1.6 \quad 1.7 \quad 1.2)\cdot kip$

Reinforcement

$B_{3.1} = "\#6"$ $B_{3.1.ref} = "Wall\ Vert.\ Perim.\ Reinf."$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = "\#\ Wall\ Vert.\ Perim.\ Bars"$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.2} = "\#4"$ $B_{3.2.ref} = "Wall\ Top.\ Perim.\ Reinf."$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = "\#\ Wall\ Top\ Perim.\ Bars"$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.3} = "\#6"$ $B_{3.3.ref} = "EW\ Bot\ Perim.\ Reinf."$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = "\#\ EW\ Bot\ Perim.\ Bars"$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = "Bar\ Diam."$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.4} = "\#4"$ $B_{3.4.ref} = "SW\ Bot\ Perim.\ Reinf."$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = "\#\ SW\ Bot\ Perim.\ Bars"$

$D_{3.4} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17 \cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5 \cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5 \cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{ewc} = 1.7 \cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5}) \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{h.e} = 1.6 \cdot in$

$a_{strip} := \frac{A_{3.5} \cdot f_y}{0.85 \cdot f'_c \cdot t_w}$ $a_{strip} = 2.4 \cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot L_{ew.mn}$ $A_{mn.e} = 0.079 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.h.e} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min} \cdot t_{slab} \cdot h_{h7.e}$ $A_{mn.strip} = 0.068 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w \cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w \cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w \cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

$chk_{c.1} := \text{if}(A_{mn.e} \le \min(A_{3.1}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.1} = \text{"OK"}$

$chk_{c.2} := \text{if}(A_{mn.e} \le \min(A_{3.2}, A_{3.5}), \text{"OK"}, \text{"NG"})$ $chk_{c.2} = \text{"OK"}$

$chk_{c.3} := \text{if}(A_{mn.strip} \le A_{3.5}, \text{"OK"}, \text{"NG"})$ $chk_{c.3} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{ewc} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{h.e} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{strip} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi Mn_{ewc} := \phi_{ewc} \cdot \min(A_{3.1}, A_{3.5}) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min(A_{3.2}, A_{3.5}) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref}$ = "Flexural Strength, +ve, Sect (22.3)"

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{ewc.1} := 8 \cdot \lambda \cdot \lambda_{s.ewc} \cdot \rho_{ewc}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975 \cdot kip$ $Vc_{ewc.2} = 19.387 \cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V \cdot \min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2 \cdot \left(t_w \cdot d_{ewc}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{ewc} = 3.7 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8 \cdot \lambda \cdot \lambda_{s.h.e} \cdot \rho_{h.e}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663 \cdot kip$ $Vc_{h.e.2} = 17.676 \cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V \cdot \min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.e} = 3.5 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8 \cdot \lambda \cdot \lambda_{s.strip}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347 \cdot kip$ $Vc_{strip.2} = 26.799 \cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V \cdot \min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.33 & 0.36 & 0.36 & 0.2 & 0.23 & 0.25 \\ 0.35 & 0.38 & 0.38 & 0.21 & 0.25 & 0.26 \\ 0.37 & 0.4 & 0.4 & 0.23 & 0.26 & 0.28 \\ 0.39 & 0.42 & 0.43 & 0.24 & 0.27 & 0.29 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

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$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{strip} = \begin{pmatrix} 0.18 & 0.18 & 0.18 & 0.11 & 0.13 & 0.14 \\ 0.19 & 0.19 & 0.19 & 0.12 & 0.14 & 0.14 \\ 0.2 & 0.2 & 0.2 & 0.13 & 0.14 & 0.15 \\ 0.16 & 0.14 & 0.14 & 0.13 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$\left(V_{h0}\ V_{h1}\ V_{h2}\ M_{h0}\ M_{h1}\ M_{h2}\right)$$

$$Uny_{hdr} = \begin{pmatrix} 0.47 & 0.75 & 0.82 & 0.18 & 0.22 & 0.24 \\ 0.5 & 0.8 & 0.87 & 0.19 & 0.24 & 0.26 \\ 0.53 & 0.84 & 0.92 & 0.2 & 0.25 & 0.27 \\ 0.55 & 0.89 & 0.97 & 0.21 & 0.26 & 0.29 \end{pmatrix} \quad L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (3.8\ \ 4.1\ \ 4.3\ \ 4.5)\cdot kip$ $Vuh1^T = (3.8\ \ 4.1\ \ 4.3\ \ 4.5)\cdot kip$ $Vuh2^T = (3.8\ \ 4.1\ \ 4.3\ \ 4.5)\cdot kip$

Wall Plates: $No_{plates} := 1$ $V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$ $V_{u.Floor.PLh0}^T = (5.9\ \ 6.3\ \ 6.6\ \ 7)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$ $V_{u.Floor.PLh1}^T = (6.6\ \ 7\ \ 7.3\ \ 7.7)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}}$ $V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$ $V_{u.Floor.PLh2}^T = (7\ \ 7.4\ \ 7.8\ \ 8.3)\cdot kip$

$A_{cv.b.3b.1}^T = (1456\ \ 1552\ \ 1648\ \ 1744)\cdot in^2$

$\rho_{b.2}^T = (0.00044\ \ 0.00041\ \ 0.00039\ \ 0.00037)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \Longrightarrow 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force

$v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.1.2h0}^T = (3.8\ \ 4.1\ \ 4.3\ \ 4.5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.2.2h1}^T = (4.3\ \ 4.5\ \ 4.8\ \ 5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.3.2h2} := \dfrac{V_{u.Wall.PLh2}}{\left[\left(L_{ew.mn}\right) - 3.5in\right]}$ $v_{u.5d.3.2h2}^T = (4.5\ \ 4.8\ \ 5.1\ \ 5.4)\cdot\dfrac{kip}{ft}$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.2.2h1}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T, \frac{v_{u.5d.3.2h2}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}^T\right)^T \qquad Hwall := \left(H_{wall_0}\ \ H_{wall_1}\ \ H_{wall_2}\right)$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$Hwall = (9.00 \quad 10.00 \quad 10.67)\,ft$

$$Unity_{5d1.2.3} = \begin{pmatrix} 59.56 & 66.18 & 70.59 \\ 63.06 & 70.07 & 74.74 \\ 66.55 & 73.95 & 78.88 \\ 70.05 & 77.83 & 83.29 \end{pmatrix} \cdot \%\ L = \begin{pmatrix} 30 \\ 32 \\ 34 \\ 36 \end{pmatrix} ft$$

$chk_{5d.1} := if\left(1.0 \geq \max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4 \cdot in$ Wall Thickness $w_{plate} = 16 \cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \\ 4 \cdot t_w + w_{plate} \end{pmatrix} \quad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix} \cdot in \quad b_{eff.5d.4} := b_{eff.5d_0} \quad b_{eff.5d.5} := b_{eff.5d_1} \quad b_{eff.5d.6} := b_{eff.5d_2}$$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$	$b_{5d.4} = 8.3\,ft$	For Wall Panel Height of
$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.4} := H_{wall_0} - t_{floor}$	$l_{c.5d.4} = 8.3\,ft$	$H_{wall_0} = 9\,ft$
$a_{5d5} := W_{ew}$	$a_{5d5} = 11.7\,ft$	$b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$	$b_{5d.5} = 9.3\,ft$	For Wall Panel Height of
$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.5} := H_{wall_1} - t_{floor}$	$l_{c.5d.5} = 9.3\,ft$	$H_{wall_1} = 10\,ft$
$a_{5d6} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$	$b_{5d.6} = 10\,ft$	For Wall Panel Height of
$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$	$a\_b_{5d.6} = 1.2$	$l_{c.5d.6} := H_{wall_2} - t_{floor}$	$l_{c.5d.6} = 10\,ft$	$H_{wall_2} = 10.7ft$

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \quad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft \quad P_{5.5} := \max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right) \quad P_{5.5} = 53.3 \cdot psf$$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4} \cdot P_{5.5} \cdot \left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 15.08 \cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6} \cdot 1\,ft$ $S_{5d.4} = 32 \cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4} \cdot S_{5d.4}$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

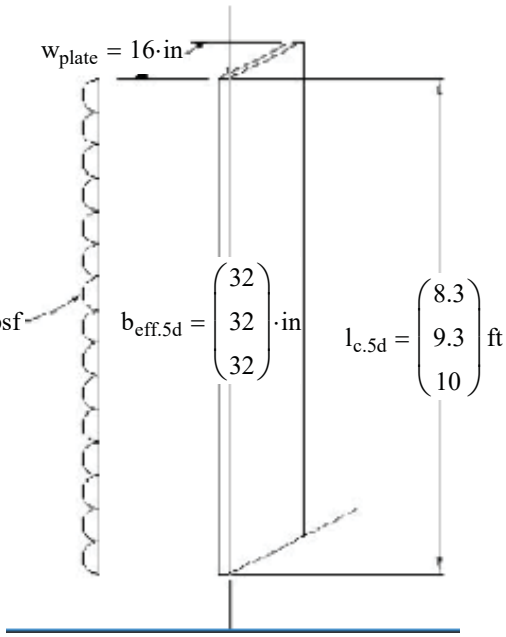
Assume only (1) end wall for resistance to lateral load $P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}{}^T = (11.8 \quad 12.5 \quad 13.2 \quad 13.9) \cdot kip$ $P_{5d.4max} := \max\left(2 \cdot V_{u.Wall.PLh0}\right)$ $P_{5d.4max} = 13.9 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}{}^T = (5.9 \quad 6.3 \quad 6.6 \quad 7) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \frac{\beta_{5d.5} \cdot P_{5.5} \cdot \left(l_{c.5d.5}\right)^2}{\left(t_w\right)^2} = 18.92 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \frac{\beta_{5d.6} \cdot P_{5.5} \cdot \left(l_{c.5d.6}\right)^2}{\left(t_w\right)^2} = 17.33 \cdot ksf$



$S_{5d.5} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15.5 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \frac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$

$P_{5d.5}^T = (13.1 \quad 13.9 \quad 14.7 \quad 15.5) \cdot kip$

$V_{u.Floor.PLh1}^T = (6.6 \quad 7 \quad 7.3 \quad 7.7) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^T = (14 \quad 14.8 \quad 15.7 \quad 16.5) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.5 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^T = (7 \quad 7.4 \quad 7.8 \quad 8.3) \cdot kip$

$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} = 231.8 \cdot kip \cdot in$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \frac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^T = (11.5 \quad 12.2 \quad 12.9 \quad 13.5) \cdot \%$

$chk_{5d.3} := if(1.0 \geq 2 \max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} = 288.5 \cdot kip \cdot in$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

$chk_{5d.4} := if\left(P_{5d.5max} \cdot \frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$\phi P_{5d.5} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.5} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.5}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9 \cdot kip$

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^T = (15.3 \quad 16.2 \quad 17.1 \quad 18) \cdot \%$

$chk_{5d.5} := if(1.0 \geq 2 \max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} = 330.8 \cdot kip \cdot in$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$ $chk_{5d.6} := if\left(P_{5d.6max} \cdot \frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.6} \cdot \left[1 - \left(\frac{k_{5.5} \cdot l_{c.5d.6}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7 \cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^T = (19 \quad 20.1 \quad 21.2 \quad 22.4) \cdot \%$

$chk_{5d.7} := if\left(1.0 \geq 2 \max\left(Unity_{5d.4}\right), "OK", "NG"\right)$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w \cdot \rho_{min.5e.1}$ $Amin_{5e.1} = 0.096 \cdot \frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^T = (1.6 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_0} = 9\ ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^T = (2.4 \quad 2.3 \quad 2.3 \quad 2.3) \cdot kip$

$V_{L.ewh0}^T = (1.2 \quad 1.3 \quad 1.4 \quad 1.5) \cdot kip$

$Unity_{5e.1.h0}^T = (52.6 \quad 56.4 \quad 60.3 \quad 64.4) \cdot \%$ $chk_{5e.1} := if\left(1.0 \geq \max\left(Unity_{5e.1.h0}\right), "OK", "NG"\right)$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^T = (1.6 \quad 1.6 \quad 1.5 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_1} = 10\ ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^T = (2.3 \quad 2.3 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh1}^T = (1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h1}^T = (57.8 \quad 62.2 \quad 66.6 \quad 71.2) \cdot \%$ $chk_{5e.2} := if\left(1.0 \geq \max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^T = (1.6 \quad 1.5 \quad 1.5 \quad 1.5)$

$L^T = (30 \quad 32 \quad 34 \quad 36)\ ft$

$H_{wall_2} = 10.7\ ft$ $Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^T = (2.3 \quad 2.2 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh2}^T = (1.4 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^T = (59.5 \quad 64.1 \quad 68.8 \quad 73.9) \cdot \%$ $chk_{5e.3} := if\left(1.0 \geq \max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^T = (11.8 \quad 12.5 \quad 13.2 \quad 13.9) \cdot kip$

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$Mu_{5d.4} = 3.35 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \quad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.4} = 15.7 \quad d_{5e.4} := \frac{t_w}{2} \quad d_{5e.4} = 2 \cdot in \quad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'^{T}_{5e.4} = (0.28 \quad 0.27 \quad 0.25 \quad 0.24) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \quad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}} \right] \right]$$

$P_{d.5e.4}^{T} = (271.2 \quad 273.7 \quad 275.9 \quad 277.9) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \quad Unity_{5e.4}^{T} = (4.4 \quad 4.6 \quad 4.8 \quad 5) \cdot \%$

$chk_{5e.4} := if(1.0 \geq \max(Unity_{5e.4}), "OK", "NG")$ $chk_{5e.4} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{5d.5}^{T} = (13.1 \quad 13.9 \quad 14.7 \quad 15.5) \cdot kip$

$Mu_{5d.5} = 4.2 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \quad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.5} = 15.7 \quad d_{5e.5} := \frac{t_w}{2} \quad d_{5e.5} = 2 \cdot in \quad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'^{T}_{5e.5} = (0.32 \quad 0.3 \quad 0.29 \quad 0.27) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \quad x_{5e.5} = 159.1 \cdot kip$

$$P_{d.5e.5} := x_{5e.5} \cdot \left[\left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}} \right] \right]$$

$P_{d.5e.5}^{T} = (265.4 \quad 268.2 \quad 270.7 \quad 273) \cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}} \quad Unity_{5e.5}^{T} = (5 \quad 5.2 \quad 5.4 \quad 5.7) \cdot \%$

$chk_{5e.5} := if(1.0 \geq \max(Unity_{5e.5}), "OK", "NG")$ $chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (14 \quad 14.8 \quad 15.7 \quad 16.5) \cdot kip$

$Mu_{5d.6} = 3.85 \cdot kip \cdot in \quad f'_c = 4500 \cdot psi \quad \phi_c = 0.65 \quad \rho_{5e.6} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.6}} \quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85 \cdot f'_c} \quad u_{5e.6} = 15.7 \quad d_{5e.6} := \frac{t_w}{2} \quad d_{5e.6} = 2 \cdot in \quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

$e'_{5e.6}{}^{T} = (0.27 \quad 0.26 \quad 0.25 \quad 0.23)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P_{d.5e.6}{}^{T} = (272.6 \quad 275 \quad 277.1 \quad 279.2)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity_{5e.6}{}^{T} = (5.1 \quad 5.4 \quad 5.7 \quad 5.9)\cdot\%$

$chk_{5e.6} := if(1.0 \geq \max(Unity_{5e.6}), "OK", "NG")$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V_{u.Floor.PLh0}{}^{T} = (5.9 \quad 6.3 \quad 6.6 \quad 7)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix}676.3\\715.6\\741.8\end{pmatrix}\cdot plf$

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix}608.6\\644\\667.6\end{pmatrix}\cdot plf$

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APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx}{}^{T} = (676.3 \quad 715.6 \quad 741.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w_{u.5f.1.DL.mx}{}^{T} = (608.6 \quad 644 \quad 667.6)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix}634.3\\673.6\\699.8\end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{(H_{wall} - t_{floor})}{3}$

$w_{5f.1} = \begin{pmatrix}2.8\\3.1\\3.3\end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix}w_{DL.5f.1.mx}\\-w_{DL.5f.1.mx}\end{pmatrix}}{No_{plates}} = \begin{bmatrix}\begin{pmatrix}676.25\\715.58\\741.81\end{pmatrix}\\\begin{pmatrix}-676.25\\-715.58\\-741.81\end{pmatrix}\end{bmatrix}\cdot plf$

$w_{up} := \frac{|P_{wind_3}|\cdot W_{ew}}{2}$ $\quad w_{up} = 311.9\cdot plf$ $\quad w_{u.up} := w_{up}$ $\quad w_{u.up} = 311.9\cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0}\cdot\frac{W_{ew}}{2} - w_{u.up}\cdot\frac{W_{ew}}{2} + Embed_{5f.1}$ $\quad P_{5f.2.h0} = (8 \quad 8.4 \quad 8.7 \quad 9.1)\cdot kip$

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$P_{5f.2.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$ $P_{5f.2.h1} = (8.3 \quad 8.6 \quad 9 \quad 9.3) \cdot kip$

$P_{5f.2.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$ $P_{5f.2.h2} = (8.4 \quad 8.8 \quad 9.1 \quad 9.5) \cdot kip$

Tension Side

$P_{5f.3.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h0} = (-3.8 \quad -4.1 \quad -4.5 \quad -4.8) \cdot kip$

$P_{5f.3.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h1} = (-3.6 \quad -3.9 \quad -4.3 \quad -4.6) \cdot kip$

$P_{5f.3.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$ $P_{5f.3.h2} = (-3.4 \quad -3.8 \quad -4.1 \quad -4.4) \cdot kip$

$W_{u.5f.1} := \left| \max\left(\left| P_{W.c} \right| , \left| P_{L.c} \right| \right) \right| = 53.3 \cdot psf$ $Mu_{5f.1} := \frac{W_{u.5f.1} \cdot (H_E - 8in)^2}{8} = \begin{pmatrix} 6.2 \\ 7.7 \\ 8.8 \end{pmatrix} \cdot \frac{kip \cdot in}{ft}$

$d_{5.4} := 0.5 \cdot t_w$ $d_{5.4} = 2 \cdot in$ $x_{5f.2} := 0.00349$ Initial Guess for reinforcement ratio

$$f(x_{5f.2}) := \frac{\frac{\max(Mu_{5f.1})}{\left(\frac{kip \cdot ft}{ft}\right)}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d_{5.4}}{in}\right)^2 \cdot (1 - 8.85 \cdot x_{5f.2})} - x_{5f.2} \qquad \rho_{5f.2} := \frac{root(f(x_{5f.2}), x_{5f.2})}{ft} \qquad \rho_{5f.2} = 0.00349 \cdot \frac{1}{ft}$$

Reinforcement Ratio Required for out of plane Wind

$A_{s.req1.5f.2.} := \rho_{5f.2} \cdot 12in \cdot d_{5.4}$ $A_{s.req1.5f.2.} = 0.084 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for out of plane Wind

$$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9 \cdot f_y \cdot \frac{W_{ew}}{2}} \right)$$

$A_{s.req2.5f.2} = 0.01533 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for Tension Force from Uplift

$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2}$ $A_{req.5f.2} = 0.099 \cdot \frac{in^2}{ft}$ Total Area of Reinforcement Required

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ Area of Steel Provided

$chk_{5.8} := if(A_{req.5f.2} \le A_{s.4a}, "OK", "NG")$ $chk_{5.8} = "OK"$

Check P-Δ Effects

$E_{c.spec} = 2991 \cdot ksi$ $I_{g.5g.1} := \frac{1}{12} \cdot 12in \cdot (t_w)^3$ $I_{g.5g.1} = 64 \cdot in^4$

$P_{cr.5g.1} := \frac{\pi^2 \cdot E_{c.spec} \cdot I_{g.5g.1}}{H_E^2}$ $P_{cr.5g.1} = \begin{pmatrix} 146 \\ 119.5 \\ 105.6 \end{pmatrix} \cdot kip$ $Ma_{g.2} := \frac{\left| \max\left(\left| P_{W.c} \right| , \left| P_{L.c} \right| \right) \right| \cdot (H_{wall})^2}{8} \cdot 12in$ $Ma_{g.2} = \begin{pmatrix} 539.3 \\ 665.9 \\ 757.6 \end{pmatrix} \cdot lb \cdot ft$

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APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$Mb_{5g.2} := Ma_{g.2} \quad Mb_{5g.2} = \begin{pmatrix} 0.5 \\ 0.7 \\ 0.8 \end{pmatrix} \cdot kip \cdot ft \qquad C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}} \qquad C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}} \qquad \delta_{ns.5.6} = \begin{pmatrix} 1.1 \\ 1.2 \\ 1.2 \end{pmatrix}$$

$$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix} \qquad Ma_{g.3} = \begin{pmatrix} 619.7 \\ 791.2 \\ 923 \end{pmatrix} \cdot lb \cdot ft \qquad f_r = 427.6 \cdot psi \qquad M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}} \qquad M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking\ Occurs", "Cracking\ Doesn't\ Occur"\right)$ $\qquad chk_{5.9} = "Cracking\ Doesn't\ Occur"$

$$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}} \qquad \Delta_{5g.1} = \begin{pmatrix} 0.18 \\ 0.23 \\ 0.26 \end{pmatrix} \cdot in \qquad Mu_{5.5} := Mu_{5f.1} \cdot ft \qquad Mu_{5.5} = \begin{pmatrix} 6.2 \\ 7.7 \\ 8.8 \end{pmatrix} \cdot kip \cdot in$$

$$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$$

$$Mu_{5.6} = (10.7 \quad 10.8 \quad 10.9 \quad 11) \cdot kip \cdot in$$

$$P_{5.8} := Embed_{5f.1}$$

$$P_{5.8} = (5.9 \quad 6.3 \quad 6.6 \quad 7) \cdot kip$$

$$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}} \qquad e`_{5.6} = (1.8 \quad 1.7 \quad 1.7 \quad 1.6) \cdot in$$

$$u_{5.} := \frac{f_y}{0.85 \cdot f`_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f`_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

$$\rho_{5.8} := \frac{A_{s.4a}}{\max\left(w_{5f.1}\right) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}} + \sqrt{\left(1 - \frac{\max\left(e`_{5.6}\right)}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max\left(e`_{5.6}\right) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 276.3 \\ 309.5 \\ 331.6 \end{pmatrix} \cdot kip$$

$chk_{5.10} := if\left(\max\left(P_{5.8}\right) \le \min\left(\phi P_{5.6}\right), "OK", "NG"\right)$ $\qquad chk_{5.10} = "OK"$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 37.6 \\ 37.5 \\ 37.4 \\ 37.3 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 30 ' "} \\ \text{"Min Flr Uniform Weight, for L = 32 ' "} \\ \text{"Min Flr Uniform Weight, for L = 34 ' "} \\ \text{"Min Flr Uniform Weight, for L = 36 ' "} \end{pmatrix}$$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$wg_{f.mx} = \begin{pmatrix} 41.6 \\ 41.5 \\ 41.4 \\ 41.3 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$$ Wall Weight

$$w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf$$ Wall Misc. Weight

$DL_{r.mx} = 53.7 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 46.7 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -53.48 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 46 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.194$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

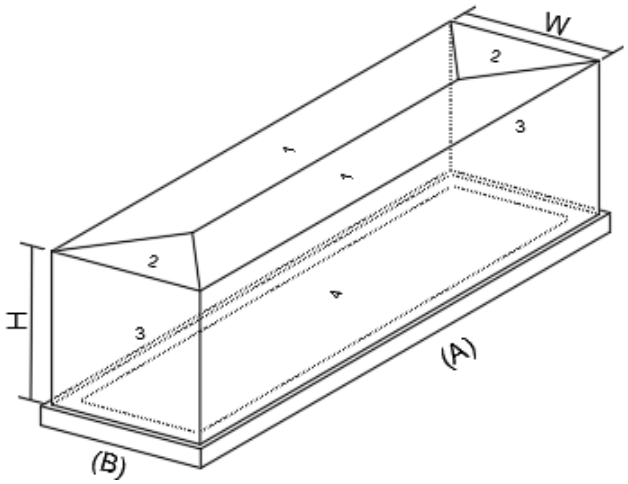
$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.155$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (364 \quad 388 \quad 412 \quad 436)\ ft^2$

$A_f^T = (320.99 \quad 342.9 \quad 364.82 \quad 386.74)\ ft^2$

$L_{wall}^T = (81.92 \quad 85.92 \quad 89.92 \quad 93.92)\ ft$



$$DL_{A.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) \ldots + \frac{W_f}{2} \cdot (wgt_{flr}) + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2} \cdot (DL_{r.mn}) + \min(wgt_{wall.plf}) + \frac{W_f}{2} \cdot (wgt_{flr}) \ldots + \min(wgt_{wall.plf}) \cdot \frac{W_{ew}}{L_f}$$

$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right) \dots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 49.31\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right) \dots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 42.07\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor} \dots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right] \dots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}\right] \qquad \begin{matrix} z_{mx} = 66.88\cdot in \\ z_{mx} = 5.57\cdot ft\end{matrix}$$

$ftg_{exp} := 36in$ Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.65\,ft$ Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\,ft$ Moment Arm for Floor Level

$DL_{A.f0.mx}^{T} = (1029.8 \quad 1029.3 \quad 1028.8 \quad 1028.5)\cdot plf$

$DL_{A.f0.mn}^{T} = (981 \quad 971.5 \quad 963.2 \quad 955.8)\cdot plf$

$DL_{B.f0.mx}^{T} = (814.9 \quad 814.4 \quad 814 \quad 813.6)\cdot plf$

$DL_{B.f0.mn}^{T} = (794.2 \quad 784.7 \quad 776.3 \quad 768.9)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1425 \quad 1424 \quad 1424 \quad 1423)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (981 \quad 972 \quad 963 \quad 956)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (795 \quad 795 \quad 794 \quad 794)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (566 \quad 566 \quad 566 \quad 566)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C}$ $\qquad E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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$E_{O.A.E.f0.C}^{T} = (254.5 \;\; 254.4 \;\; 254.3 \;\; 254.3)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-254.5 \;\; -254.4 \;\; -254.3 \;\; -254.3)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (53.1 \;\; 49.7 \;\; 46.7 \;\; 44.1)\cdot plf$

$E_{O.B.E.f0.T}^{T} = (-53.1 \;\; -49.7 \;\; -46.7 \;\; -44.1)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (242.3 \;\; 242.2 \;\; 242.1 \;\; 242)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-242.3 \;\; -242.2 \;\; -242.1 \;\; -242)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (51.6 \;\; 48.3 \;\; 45.4 \;\; 42.8)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-51.6 \;\; -48.3 \;\; -45.4 \;\; -42.8)\cdot plf$

$E_{O.A.E.f2.C}^{T} = (230 \;\; 229.9 \;\; 229.9 \;\; 229.8)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-230 \;\; -229.9 \;\; -229.9 \;\; -229.8)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (50.1 \;\; 46.9 \;\; 44 \;\; 41.5)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-50.1 \;\; -46.9 \;\; -44 \;\; -41.5)\cdot plf$

$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$

$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$

$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$

$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$

$E_{8p.A.f0}^{T} = (752.9 \;\; 752.6 \;\; 752.4 \;\; 752.2)\cdot plf$

$E_{8n.A.f0}^{T} = (-216.4 \;\; -216.7 \;\; -216.9 \;\; -217.1)\cdot plf$

$E_{8p.B.f0}^{T} = (331.1 \;\; 327.7 \;\; 324.6 \;\; 321.9)\cdot plf$

$E_{8n.B.f0}^{T} = (-31.1 \;\; -27.7 \;\; -24.7 \;\; -22.1)\cdot plf$

$E_{8p.A.f1}^{T} = (716.7 \;\; 716.4 \;\; 716.2 \;\; 716)\cdot plf$

$E_{8p.B.f1}^{T} = (321.6 \;\; 318.2 \;\; 315.3 \;\; 312.7)\cdot plf$

$E_{8n.A.f1}^{T} = (-192 \;\; -192 \;\; -191.9 \;\; -191.9)\cdot plf$

$E_{8n.B.f1}^{T} = (-22.4 \;\; -19.1 \;\; -16.2 \;\; -13.6)\cdot plf$

$E_{8p.B.f2}^{T} = (312.1 \;\; 308.8 \;\; 306 \;\; 303.4)\cdot plf$

$E_{8n.A.f2}^{T} = (-182.5 \;\; -182.4 \;\; -182.3 \;\; -182.3)\cdot plf$

$E_{8p.A.f2}^{T} = (680.5 \;\; 680.2 \;\; 680 \;\; 679.8)\cdot plf$

$E_{8n.B.f2}^{T} = (-21.8 \;\; -18.6 \;\; -15.8 \;\; -13.3)\cdot plf$

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$LL_{r.A.} := \frac{W_r}{2} \cdot (w_{llr})$ $LL_{r.A.} = 210 \cdot plf$ $LL_{r.B.} := \frac{W_r}{6} \cdot (w_{llr})$ $LL_{r.B.} = 70 \cdot plf$

$LL_{A.f0} := \frac{W_f}{2} \cdot (w_{llf.0})$ $LL_{A.f0} = 1369.79 \cdot plf$ $LL_{B.f0} := s_{t.rib} \cdot (w_{llf.0})$ $LL_{B.f0} = 500 \cdot plf$

$LL_{A.f1} := \frac{W_f}{2} \cdot (w_{llf.1})$ $LL_{A.f1} = 1095.83 \cdot plf$ $LL_{B.f1} := s_{t.rib} \cdot (w_{llf.1})$ $LL_{B.f1} = 400 \cdot plf$

$LL_{A.f2} := \frac{W_f}{2} \cdot (w_{llf.2})$ $LL_{A.f2} = 821.88 \cdot plf$ $LL_{B.f2} := s_{t.rib} \cdot (w_{llf.2})$ $LL_{B.f2} = 300 \cdot plf$

$SL_A := \frac{W_r}{2} \cdot (p_f)$ $SL_A = 190.51 \cdot plf$ $SL_B := \frac{W_r}{6} \cdot (p_f)$ $SL_B = 63.5 \cdot plf$

$W_{up.A} := \frac{W_r}{2} \cdot P_{wind_3}$ $W_{up.A} = -320.86 \cdot plf$ $W_{up.B} := \frac{W_r}{6} \cdot P_{wind_3}$ $W_{up.B} = -106.95 \cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat} \cdot H_{wall} \cdot \left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $W_{OT.A.C} = 618.5 \cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat} \cdot H_{wall} \cdot \left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $W_{OT.B.C} = 231.4 \cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $W_{OT.A.T} = -618.5 \cdot plf$ $W_{OT.B.T} := -W_{OT.B.C}$ $W_{OT.B.T} = -231.4 \cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $W_{A.mx} = 297.6 \cdot plf$ $W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $W_{B.mx} = 124.4 \cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $W_{A.mn} = -939.4 \cdot plf$ $W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $W_{B.mn} = -338.3 \cdot plf$

b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \quad Load_{f0} = \begin{pmatrix} 1029.76 \\ 955.76 \\ 1369.79 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 752.86 \\ -217.12 \end{pmatrix} \cdot plf$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$\text{ASD} = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f0} := asd \cdot Load_{f0} \quad LC_{8.f0} = \begin{pmatrix} 2400 \\ 1240 \\ 2267 \\ 1208 \\ 466 \\ 2385 \\ 1458 \\ 752 \\ 10 \\ 1557 \\ 878 \\ 2595 \\ 2086 \\ 1100 \\ 421 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 12.46 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 13 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f0} := \max(14in, w_{8.f0.min})$ $\quad Fdn_{W.f0} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 14 \cdot in$

consider $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \quad Load_{f1} = \begin{pmatrix} 1029.76 \\ 955.76 \\ 1095.83 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 716.67 \\ -192.04 \end{pmatrix} \cdot plf$$

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f1} := asd \cdot Load_{f1} \quad LC_{8.f1} = \begin{pmatrix} 2126 \\ 1240 \\ 2062 \\ 1208 \\ 466 \\ 2180 \\ 1252 \\ 752 \\ 10 \\ 1531 \\ 895 \\ 2371 \\ 1894 \\ 1075 \\ 439 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 11.38 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 12 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f1} := \max(14in, w_{8.f1.min})$ $\quad Fdn_{W.f1} = 14 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

consider $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \quad Load_{f2} = \begin{pmatrix} 1029.76 \\ 955.76 \\ 821.88 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 680.48 \\ -182.46 \end{pmatrix} \cdot plf$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.com

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \quad LC_{8.f2} := asd \cdot Load_{f2} \quad LC_{8.f2} = \begin{pmatrix} 1852 \\ 1240 \\ 1856 \\ 1208 \\ 466 \\ 1974 \\ 1047 \\ 752 \\ 10 \\ 1506 \\ 902 \\ 2146 \\ 1693 \\ 1050 \\ 446 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 10.3 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 11 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f2} := \max(12in, w_{8.f2.min})$ $\quad Fdn_{W.f2} = 12 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, &

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 12 \cdot in$

c. Slab on Grade Foundation

1.) Criteria: use slab on grade. $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $\quad Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

$$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.long} \qquad A_{s.long.prov} = 0.39 \cdot in^2$$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if\left(A_{s.long.prov} \geq A_{s.long}, "OK", "NG"\right)$ $chk_{8.1} = "OK"$

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp} \qquad A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4 \qquad Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^{2}}{4}\right) \cdot Qty_{bar.temp}$$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if\left(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG"\right)$ $chk_{8.2} = "OK"$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 46.708 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 53.7 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 35 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 46 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 35.8 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 32.3 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 53.5 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 2.99 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 2.99 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 9.226 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.021 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.021 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.395 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.395 \cdot klf$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

SEISMIC LOAD PARAMETERS: *For worst case seismic load in E-W direction, look at longest, widest shelter.*

Seismic Coefficient

$C_s = 0.194$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 27.23\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 24.18\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 34.82\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 29.38\cdot kip$

Total Seismic Shear at Roof

$$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$$ $V_{smaxW\_rf} = 8.675\cdot kip$

$$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$$ $V_{sminW\_rf} = 7.553\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$$ $v_{EW\_smaxW} = 0.145\cdot klf$

$$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$$ $v_{EW\_sminW} = 0.105\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$$ $v_{NS\_smaxW} = 0.372\cdot klf$

$$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$$ $v_{NS\_sminW} = 0.324\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$$ Modification Factor for Eccentric Loading

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

k_c Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix} =$ ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.047 \cdot klf$ Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.372 \cdot klf$ Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.16 \cdot klf$ Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.182 \cdot klf$ Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.094 \cdot klf$ Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.395 \cdot klf$ Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 7$

$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1}$ $s_{REW} = 18.67\cdot in$

$s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$ $s_{EWR} = 19.33\cdot in$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$ $Vu_{o.s.REW} = 0.026\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$ $Vu_{i.s.REW} = 0.675\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$ $Vu_{o.w.REW} = 0.171\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$ $Vu_{i.w.REW} = 0.718\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.REW} := F_{7.1}\cdot max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$

$Tu_{T.REW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$ $Vu_{o.s.EWR} = 0.026\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$ $Vu_{i.s.EWR} = 0.699\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.EWR} := F_{7.2}\cdot max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$ $Vu_{o.w.EWR} = 0.177\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$ $Vu_{i.w.EWR} = 0.743\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$ $Tu_{T.EWR} = 0.177\cdot kip$ 0.9DL + 1.0W T, roof uplift

▶ Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 4 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 90 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 4.57 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 4 \cdot in$ $h_{min.7.1a} = 4 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 107 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 3 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min(0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1})$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.307 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.368 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.026$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.298 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.362 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.1} = 0.205 \cdot in$$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.2} = 0.23 \cdot in$$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2} \qquad l_{w\_Rqrd.REW.3} = 2.5 \cdot in$$

$$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right) \qquad l_{w\_reqd.REW} = 2.5 \cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.047\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.145\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.321\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.322\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.188\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.021\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 48\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.3} := \text{if}(s_{L.7.3} = 0in, s_{w.7.3}, \text{if}(s_{w.7.3} = 0in, s_{L.7.3}, \min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := \text{if}(s_{L.7.4} = 0in, s_{w.7.4}, \text{if}(s_{w.7.4} = 0in, s_{L.7.4}, \min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.219\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.202\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 0.878\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.097\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0.144\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.219\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.202\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 0.878\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.097\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0.144\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0.033$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.157 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.326 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0.021$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.151 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.307 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.087\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.262\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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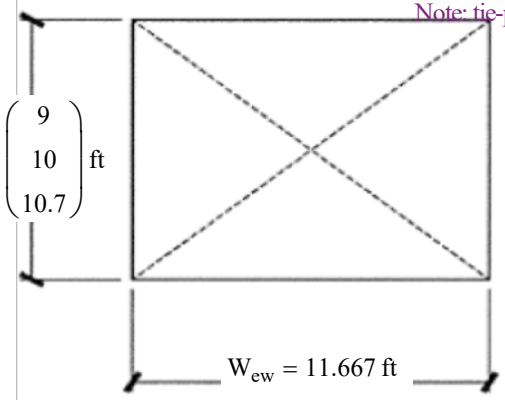
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 5$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}$ ft

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (20.5 \quad 23.5 \quad 25.5)\cdot in$ $s_{SWEW}^{T} = (20.5 \quad 23.5 \quad 25.5)\cdot in$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6\,ft$ Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 5.275\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{..}\cdot L_{..}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (0.669 \quad 0.628 \quad 0.591 \quad 0.559)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.007\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 1.096\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10\,ft$

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

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$M_{ot\_w.EWSW}^{T} = (59.9 \quad 62.8 \quad 65.8 \quad 68.8)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r + \frac{\max\left[\left(\omega_{wall}\right)\cdot\left[\max\left(H_{wall}\right)\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (78 \quad 80.9 \quad 83.8 \quad 86.8)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (90.9 \quad 96.2 \quad 101.5 \quad 106.8)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (-21.99 \quad -23.76 \quad -25.53 \quad -27.3)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (-12.92 \quad -15.3 \quad -17.68 \quad -20.05)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max\left(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net}\right)$ $M_{ot.EWSW} = 0\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := \text{if}\left(s_{L.7.5} = 0in, s_{w.7.5}, \text{if}\left(s_{w.7.5} = 0in, s_{L.7.5}, \min\left(s_{L.7.5}, s_{w.7.5}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := \text{if}\left(s_{L.7.6} = 0in, s_{w.7.6}, \text{if}\left(s_{w.7.6} = 0in, s_{L.7.6}, \min\left(s_{L.7.6}, s_{w.7.6}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.468\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.468\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wall (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.468 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.468 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.109$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.161$ < 1.0

$Result_{EWSW} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW} =$ "not required" < 1.2

$chk_{6.EWSW} =$ "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.111$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.29$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.194 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.194 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right)$ $l_{w\_reqd.EWSW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\ ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.649 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.108 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.023 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.372 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.395 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.165 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.094 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\ ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$M_{ot.s.FEW}^{T} = (72.4 \quad 77.2 \quad 82 \quad 86.8)\cdot kip\cdot ft$

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix}0.5\cdot(L_0)^2\\0.5\cdot(L_1)^2\\0.5\cdot(L_2)^2\\0.5\cdot(L_3)^2\end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$$

OT Moment, resisting

$M_{res.FEW}^{T} = (803.3 \quad 856.9 \quad 910.4 \quad 964)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-323.2 \quad -321.1 \quad -315.7 \quad -307.2)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = (-731 \quad -779.7 \quad -828.5 \quad -877.3)\cdot kip\cdot ft$

$M_{ot.FEW} := \max(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net})$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right] \qquad v_{ot.w.FEW} = 0\cdot kip$$

$$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right) \qquad v_{ot.s.FEW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor $\quad$ $Plate_{7.8}$ = "#2" at inside face of end wall

No. of Plates & maximum plate spacing $\quad No_{pl.FEW} = 7$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \qquad s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1}$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$s_{F\_EW} = 15.92\cdot in$ $\quad s_{EW\_F} = 17.33\cdot in$

$s_{7.7} := \text{if}(s_{L.7.7} = 0in, s_{w.7.7}, \text{if}(s_{w.7.7} = 0in, s_{L.7.7}, \min(s_{L.7.7}, s_{w.7.7}))) = 3\cdot in$ $\quad s_{7.7} = 3\cdot in$

$$F_{7.7} := 0.5 + \frac{0.5s_{7.7} + 2e_{PL.F\_EW}}{s_{7.7}} \qquad F_{7.7} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.7

$s_{7.8} := \text{if}(s_{L.7.8} = 0in, s_{w.7.8}, \text{if}(s_{w.7.8} = 0in, s_{L.7.8}, \min(s_{L.7.8}, s_{w.7.8}))) = 3\cdot in$ $\quad s_{7.7} = 3\cdot in$

$$F_{7.8} := 0.5 + \frac{0.5s_{7.8} + 2e_{PL.F\_EW}}{s_{7.8}} \qquad F_{7.8} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.8

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right) \qquad Vu_{i.s.FEW} = 1.089\cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{i.w.FEW} = 0.769\cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$$Tu_{po.s1.FEW} := F_{7.7}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right) \qquad Tu_{po.s1.FEW} = 0.02\cdot kip$$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Tu_{po.w.FEW} = 0.366\cdot kip$$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{o.9dl.FEW} = 1.136\cdot kip$$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7}\cdot\frac{v_{ot.w.FEW}}{No_{pl.FEW}} \qquad V_{ot.w.FEW} = 0\cdot kip$$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot.s.FEW} := F_{7.7}\cdot\frac{v_{ot.s.FEW}}{No_{pl.FEW}} \qquad V_{ot.s.FEW} = 0\cdot kip$$ 1.0E

$$Vu_{o.wup.FEW} := F_{7.7}\cdot\max\left[\left(1.0v_{up.w.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0\cdot kip\right]$$

$$Vu_{o.wup.FEW} = 0\cdot kip$$ 0.9D+1.0W

$$Vu_{o.sup.FEW} := F_{7.7}\cdot\max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0\cdot kip\right)$$ Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.sup.FEW} = 0\cdot kip$$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7}\cdot\left[1.2\left(v_{DL.FEW}\right) + 1.6v_{LL.FEW}\right]\cdot\frac{W_{ew}}{No_{pl.FEW}}$$

$$Vu_{o.12D.16L.FEW} = 1.851\cdot kip$$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Endwall (Floor Wall Plates)

$$Tu_{po.w.EWF} := F_{7.8}\cdot 1.0\cdot 2\cdot t_{p.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Tu_{po.w.EWF} = 0.366\cdot kip$$ pullout, wind suction at bot of wall and bot half of two sides of wall

$$Tu_{po.s2.EWF} := F_{7.8}\cdot 0.3C_s\cdot\left(t_{o.s.FEW}\cdot W_{ew}\right) \qquad Tu_{po.s2.EWF} = 0.02\cdot kip$$ 0.3E pullout, seismic

$$Vu_{i.s.EWF} := F_{7.8}\cdot\left(1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} + C_s\,\omega_{wall}\cdot\frac{\max(H_{wall})}{4}\cdot W_{ew}\right)$$

$$Vu_{i.s.EWF} = 1.089\cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w2.EWF} := F_{7.8}\cdot 1.0v_{i.w.FEW}\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{i.w2.EWF} = 0.769\cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$$Vu_{o.9dl.EWF} := F_{7.8}\cdot\left(0.9v_{DL.FEW}\right)\cdot\frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{o.9dl.EWF} = 1.136\cdot kip$$ 0.9D Shear from DL only

$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - \left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 1.005 \cdot kip$ 0.9D+1.0W

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[\left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

$Vu_{o.sdn.EWF} := F_{7.8} \cdot \left(Vu_{o.9dl.FEW} + V_{ot.s.FEW}\right)$ $Vu_{o.sdn.EWF} = 1.326 \cdot kip$ 0.9D+1.0E

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6 v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 1.851 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$

$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75 \cdot V_{cbg\_op.7.7} \\ 0.75 \cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_EW} = \min\left(\left(0.75 \cdot N_{sa.7.7} \quad 0.75 \cdot N_{cbg.7.7} \quad 0.7 \cdot N_{pn.7.7}\right)\right)$ $\phi P_{pl.F\_EW} = 8.935 \cdot kip$

Plate Summary $\dfrac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041$ or $\left[\dfrac{Vu_{o.wup.FEW}}{0.75 \cdot V_{cbg\_op.7.7}} + \dfrac{Vu_{i.w.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.157 < 1.0$

$\dfrac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041$ or $\left[\dfrac{Vu_{o.sup.FEW}}{\left(0.75 \cdot V_{cbg\_op.7.7}\right)} + \dfrac{Vu_{i.s.FEW}}{\left(0.75 \cdot V_{cbg\_ip.7.7}\right)}\right] = 0.223 < 1.0$

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$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.544 \quad < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted"

$IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $V_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719\cdot in$ $h_{min.7.8} = 4\cdot in$ $c_{a1\_op.7.8} = 6.5\cdot in$ $c_{a2\_op.7.8} = 10.5\cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90\cdot in^2$ $A_{Vco\_op.7.8} = 190\cdot in^2$ $e'_{V.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0\cdot in$ $s2_{7.8a} = 3\cdot in$ $l_{e.7.8} = 1.719\cdot in$ $h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$ $h_{min.7.8a} = 4\cdot in$

$c_{a1\_ip.7.8} = 13.5\cdot in$ $c_{a2\_ip.7.8} = 6.5\cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107\cdot in^2$ $A_{Vco\_ip.7.8} = 820\cdot in^2$ $e'_{V.7.8a} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1\cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3\cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74\cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8}\cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74\cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75\cdot V_{cbg\_op.7.8} \\ 0.75\cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix}\cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\begin{pmatrix} 0.75\cdot N_{sa.7.8} & 0.75\cdot N_{cbg.7.8} & 0.7\cdot N_{pn.7.8} \end{pmatrix}\right)$ $\phi P_{pl.EW\_F} = 4.305\cdot kip$

Plate Summary

$$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.004 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.87 \quad < 1.0$$

$$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.085 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75\cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75\cdot V_{cbg\_ip.7.8}\right)}\right] = 0.634 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75\cdot V_{cbg\_op.7.8}}\right) = 0.54 \quad < 1.0$$

$Result_{EW\_F} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F} =$ "not required" < 1.2

$chk_{6.EW\_F} =$ "OK"

Weld Between $Plate_{7.7} =$ "#1B" and $Plate_{7.8} =$ "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.1} = 0.25\cdot in$$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.2} = 0.319\cdot in$$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.3} = 0.386\cdot in$$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^{2} + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^{2} + Vu_{i.s.EWF}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.503\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.503\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$l_{w\_reqd.EWSW} = 2.5\cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

****

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f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $\quad w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $\quad v_{dl.FSW} = 0.806\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $\quad v_{ll.FSW} = 0.21\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $\quad t_{o.s.FSW} = 0.047\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $\quad v_{i.s.FSW} = 0.145\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $\quad v_{i.w.SW} = 0.0208\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $\quad v_{up.w.FSW} = 0.321\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $\quad t_{po.w.FSW} = 0.172\cdot klf$ Wind load suction on bottom of side wall

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^{T} = (399.8 \quad 450.1 \quad 503.7 \quad 560.4)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^{T} = (72.4 \quad 77.2 \quad 82 \quad 86.8)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $\quad M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^{T} = (803.3 \quad 856.9 \quad 910.4 \quad 964)\cdot kip\cdot ft$

$$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.w.FSW} = 0\cdot kip$$

$$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \qquad v_{ot.s.FSW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9}$ = "#1B" at top of floor

$Plate_{7.10}$ = "#2" at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 24\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$$s_{7.9} := \text{if}\left(s_{L.7.9} = 0in, s_{w.7.9}, \text{if}\left(s_{w.7.9} = 0in, s_{L.7.9}, \min\left(s_{L.7.9}, s_{w.7.9}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}} \qquad F_{7.9} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.9

$$s_{7.10} := \text{if}\left(s_{L.7.10} = 0in, s_{w.7.10}, \text{if}\left(s_{w.7.10} = 0in, s_{L.7.10}, \min\left(s_{L.7.10}, s_{w.7.10}\right)\right)\right) = 3\cdot in \qquad s_{7.9} = 3\cdot in$$

$$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}} \qquad F_{7.10} = 1.167$$

Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.FSW} = 0.401\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.FSW} = 0.11\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.FSW} = 0.101\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.FSW} = 0.048\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.FSW} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.FSW} = 1.45\cdot kip$ 0.9D Shear from DL only

$$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}} \qquad V_{ot\_w.FSW} = 0\cdot kip$$

$$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}} \qquad V_{ot\_s.FSW} = 0\cdot kip$$

1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\begin{bmatrix}\left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, \left(0.9v_{dl.FSW} + 1.0\cdot v_{up.w.FSW}\right)\cdot s_{max.FSW} \ldots, 0\cdot kip \\ + V_{ot\_w.FSW} \qquad\qquad + -V_{ot\_w.FSW}\end{bmatrix}$$

$Vu_{o.w.FSW} = 2.441\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$$

$Vu_{o.s.FSW} = 1.692\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$$

$Vu_{o.12D.16L.FSW} = 3.04\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.401\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.11\cdot kip$ 0.3W T, pullout seismic

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$Vu_{i.s.SWF} := F_{7.10} \cdot 0.3 v_{i.s.FSW} \cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.101 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10} \cdot 1.0 v_{i.w.SW} \cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.048 \cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := (0.9 v_{dl.FSW}) \cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.45 \cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0 \cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0 \cdot kip$

$$Vu_{o.w.SWF} := F_{7.10} \cdot \max\left[\begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, \begin{pmatrix} 0.9 v_{dl.FSW} \cdots \\ +1.0 \cdot v_{up.w.FSW} \\ +-V_{ot\_w.SWF} \end{pmatrix} \cdot s_{max.FSW} \cdots, 0 \cdot kip\right]$$

$Vu_{o.w.SWF} = 2.441 \cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.SWF} := F_{7.10} \cdot \max(V_{ot\_s.SWF} - 0.9 v_{dl.FSW} \cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9 v_{dl.FSW} \cdot s_{max.FSW}, 0 \cdot kip)$

$Vu_{o.s.SWF} = 1.692 \cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.SWF} := F_{7.10} \cdot [1.2(v_{dl.FSW}) + 1.6 v_{ll.FSW}] \cdot s_{max.FSW}$

$Vu_{o.12D.16L.SWF} = 3.04 \cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5 \cdot in$ $w_{pl.7.9} = 5 \cdot in$ $x_{Le.7.9} = 1 \cdot in$ $x_{we.7.9} = 4 \cdot in$ $t_{pl.7.9} = 0.313 \cdot in$

$h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0 \cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5 \cdot in$ $A_{SE.7.9} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $N_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.045 \quad \text{or} \quad \left[\frac{Vu_{o.w.FSW}}{\left(0.75 \cdot V_{cbg\_op.7.9}\right)} + \frac{Vu_{i.w.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.728 < 1.0$$

$$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.012 \quad \text{or} \quad \left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.52 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.894 < 1.0$$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.093 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.73 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.025 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.534 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.883 < 1.0$$

$Result_{SWF}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWF}$ = "not required" < 1.2

$chk_{6.SWF}$ = "OK"

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.1} = 0.725 \cdot in$$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.2} = 0.498 \cdot in$$

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.725 \cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.498 \cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.158 \cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min\left(w_{pl.7.9}, w_{pl.7.10}\right)}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5 \cdot in$$

$l_{w\_reqd.FSW} := \max\left(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6}\right)$ $l_{w\_reqd.FSW} = 2.5 \cdot in$

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g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $\quad wgt_r = 46.708\cdot psf$ — Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $\quad wgt_w = 39.333\cdot psf$ — Wall Dead Load

$wgt_{flr}^{T} = (37.56 \quad 37.47 \quad 37.39 \quad 37.32)\cdot psf$ — Floor Dead Load

$W_{lat} = 45.986\cdot psf$ — Wind Load

$w_{llf.0} = 250\cdot psf$ $\quad w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200\cdot psf$ $\quad w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150\cdot psf$ $\quad w_{llf.2.ref}$ = "Floor Live Load Option 2"

$A_r := L_r\cdot W_r$ $\quad A_r^{T} = (364 \quad 388 \quad 412 \quad 436)\ ft^2$ — Area of Roof/Floor

$A_f := L_f\cdot W_f$ $\quad A_f^{T} = (321 \quad 342.9 \quad 364.8 \quad 386.7)\ ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ ... \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$

Allowable Resisting Moment

$M_R^{T} = (497.3 \quad 508.2 \quad 519.1 \quad 530)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$

Overturning Moment due to Wind

$M_{OT.wind}^{T} = (120.7 \quad 128.8 \quad 136.8 \quad 144.9)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^{T} = (181.1 \quad 193.2 \quad 205.2 \quad 217.3)\cdot kip\cdot ft$

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\left[\begin{matrix} L_r\cdot W_r\cdot(wgt_r)\cdot\max(H_E) + L_r\cdot W_r\cdot(0.25w_{llr})\cdot\max(H_E) \ ... \\ +\left(2L_f\cdot\max(H_E) + 2W_{ew}\cdot\max(H_E)\right)\cdot\frac{wgt_w\cdot\max(H_E)}{2} \ ... \\ + L_f\cdot W_f\cdot wgt_{flr}\cdot\frac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot(0.25w_{llf.0})\cdot t_{floor} \end{matrix}\right]$$

Overturning Moment due to seismic

$M_{OT.seismic}^{T} = (62.855 \quad 66.327 \quad 69.799 \quad 73.27)\cdot kip\cdot ft$

$1.5\,M_{OT.seismic}^{T} = (94.283 \quad 99.49 \quad 104.698 \quad 109.905)\cdot kip\cdot ft$

$M_{ot.w.net} := (1.5M_{OT.wind} - M_R)$ $\quad M_{ot.s.net} := (1.5M_{OT.seismic} - M_R)$

$M_{ot.w.net}^{T} = (-316.2 \quad -315 \quad -313.9 \quad -312.7)\cdot kip\cdot ft$

$M_{ot.s.net}^{T} = (-403 \quad -408.7 \quad -414.4 \quad -420.1)\cdot kip\cdot ft$

$chk_{S7.15} := \text{if}(\max(M_{ot.w.net}, M_{ot.s.net}) \geq 0, \text{"NG"}, \text{"OK"})$ $\quad chk_{S7.15}$ = "OK"

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Sliding

$$WT := \begin{pmatrix} L_r \cdot W_r \cdot wgt_r + L_f \cdot \min(H_E) \cdot wgt_w \ldots \\ + 2W_{ew} \cdot \min(H_E) \cdot wgt_w + 2L_f \cdot W_f \cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^T = (142.579 \quad 144.446 \quad 146.313 \quad 148.179) \cdot kip$$

$$WT_{max} := \max\begin{bmatrix} A_r \cdot (wgt_r + w_{misc.r}) + A_f \cdot (wgt_{flr} + w_{misc.f}) \ldots \\ + 2L_f \cdot \max(H_E) \cdot (wgt_w + w_{misc.w}) \ldots \\ + 2 \cdot W_{ew} \cdot \max(H_E) \cdot wgt_w + [A_r \cdot w_{llr} + A_r \cdot (w_{llf.0})] \end{bmatrix}$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 252.21 \cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in \cdot 4in$ $A_{pad} = 16 \cdot in^2$

$S_{bp} := 3ft$

Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad} = 4.729 \cdot kip$

Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 295.6 \cdot psi$

Compressive Stress per Pad

$F_{pad} := 800psi$

Allowable Stress per pad

$F_P := 0.7 \cdot 0.85 f'_{c.fdn}$

$F_P = 1785.1 \cdot psi$

Allowable Stress of Fdn Concrete

$chk_{S7.16} := \text{if}(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, \text{"OK"}, \text{"NG"})$

$chk_{S7.16} = \text{"OK"}$

$$wt_{pad.big} := \max\left[\frac{\max(WT_{max})}{\frac{2[(\max(L) - 2 \cdot t_w) + (W_{ew})]}{S_{bp}}}\right]$$

$wt_{pad.big} = 8.049 \cdot kip$

Weight per Bearing Pad

$$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$$

$f_{pad.big} = 503.1 \cdot psi$

Compressive Stress per Pad

$chk_{S7.17} := \text{if}(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, \text{"OK"}, \text{"NG"})$

$chk_{S7.17} = \text{"OK"}$

$\mu := 0.25$

Coefficient of Friction

Wind $V_{net.w.sl} := \max\begin{bmatrix} 0kip, \begin{bmatrix} \mu \cdot [-(WT) + 1.5 |P_{wind_3}| \cdot (A_r)] \ldots \\ + 1.5W_{lat} L \cdot \max(H_E) \end{bmatrix} \end{bmatrix}$

$V_{net.w.sl} = 0 \cdot kip$

Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := \max\begin{bmatrix} 0kip, 1.5 \cdot \begin{bmatrix} 0.7C_s \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} + -\mu \cdot \begin{pmatrix} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{pmatrix} \end{bmatrix} \end{bmatrix}$

$V_{net.s.sl} = 0 \cdot kip$

Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$chk_{S7.18} := \text{if}(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, \text{"Slide Plates Req'd"}, \text{"OK"})$

$chk_{S7.18} = \text{"OK"}$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \begin{cases} \max\left(2, ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) & \text{if } \frac{ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ \max\left[2, \left(ceil\left(\frac{\max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] & \text{otherwise} \end{cases}$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (28 \quad 30 \quad 32 \quad 34)\,ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3\,ft$ on center.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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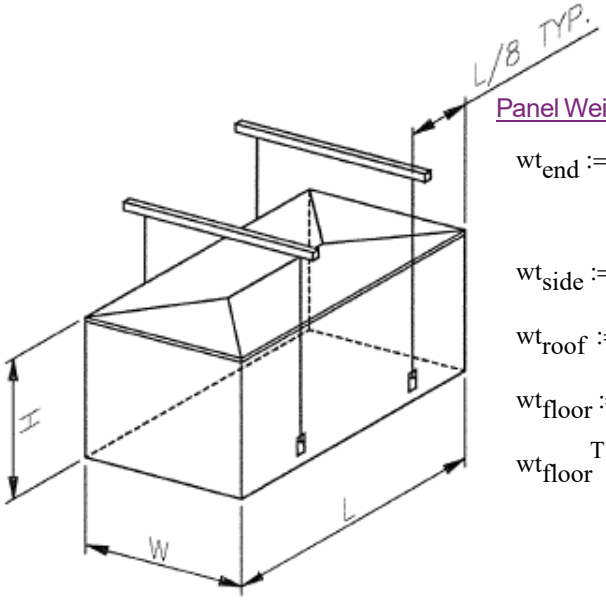
8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5115 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 438 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 561 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (438 \quad 437 \quad 436 \quad 435) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Plan No.:	MFT-4193-Standard Concrete
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The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) $M/(0.9f_yd)$			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	30.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.222	0.199
12.00	32.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.257	0.226
12.00	34.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.295	0.253
12.00	36.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.336	0.280
12.00	30.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.210	0.154
12.00	32.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.244	0.175
12.00	34.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.280	0.196
12.00	36.00	10.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.011	0.047	0.318	0.216
12.00	30.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.204	0.135
12.00	32.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.236	0.153
12.00	34.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.271	0.171
12.00	36.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.309	0.190

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Manufacturer: Modular Connections, LLC

b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

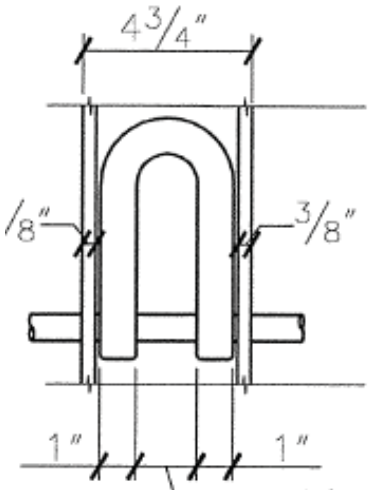
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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Occupancy:	S2
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Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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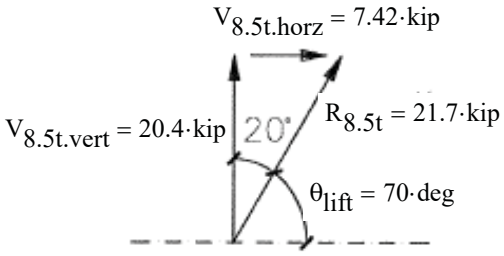
Manufacturer:	Modular Connections, LLC



$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

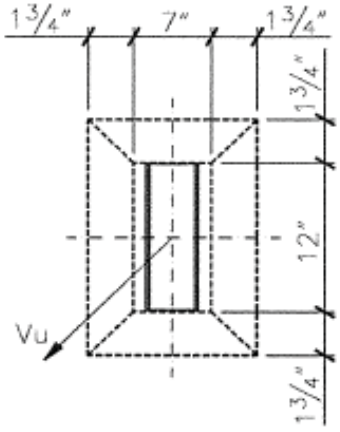
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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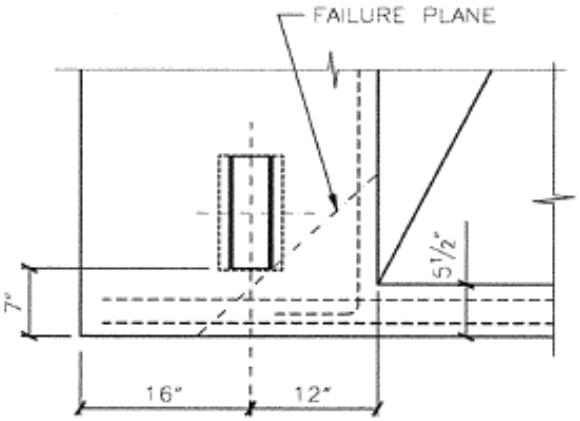
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

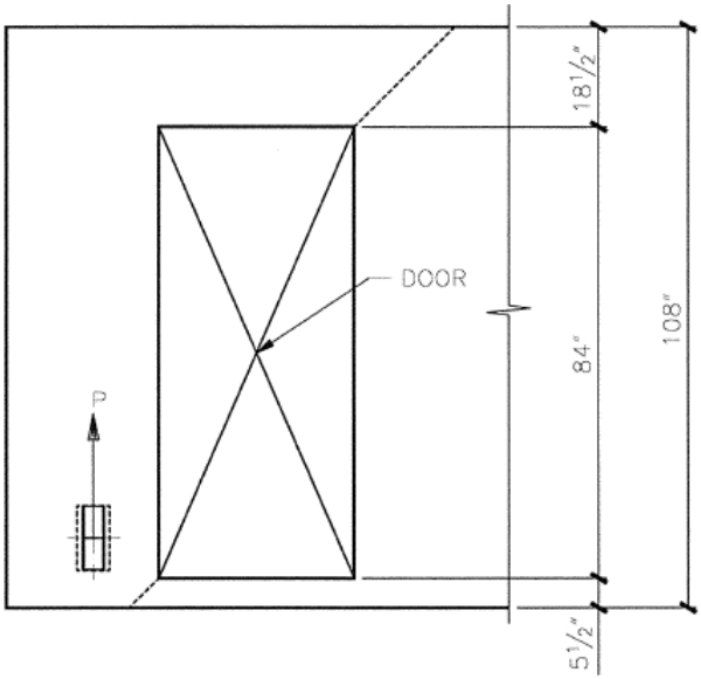
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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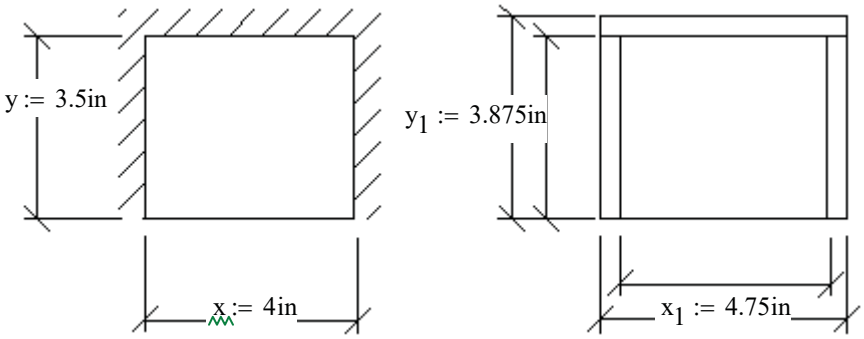
Bearing Pressure

Vertical



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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

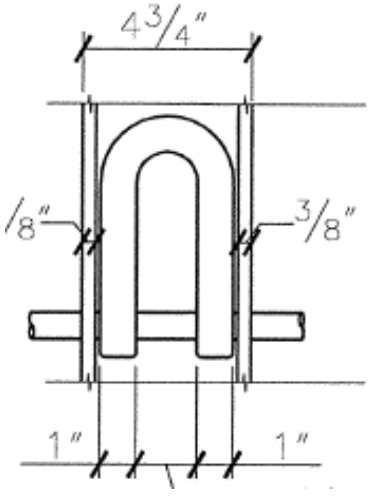
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

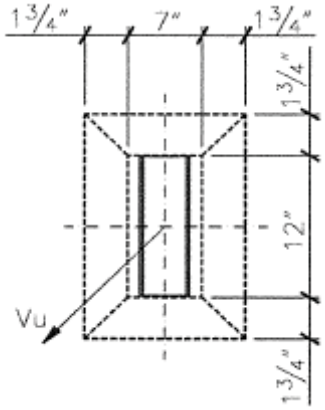
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

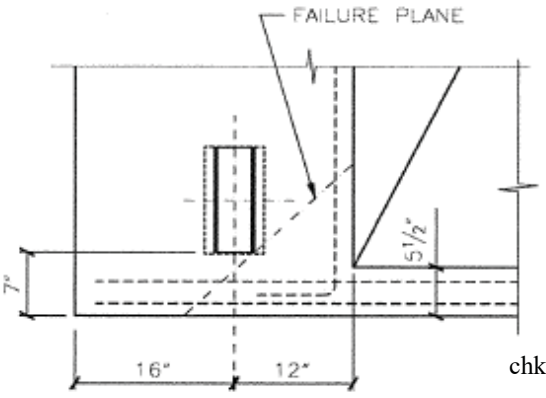
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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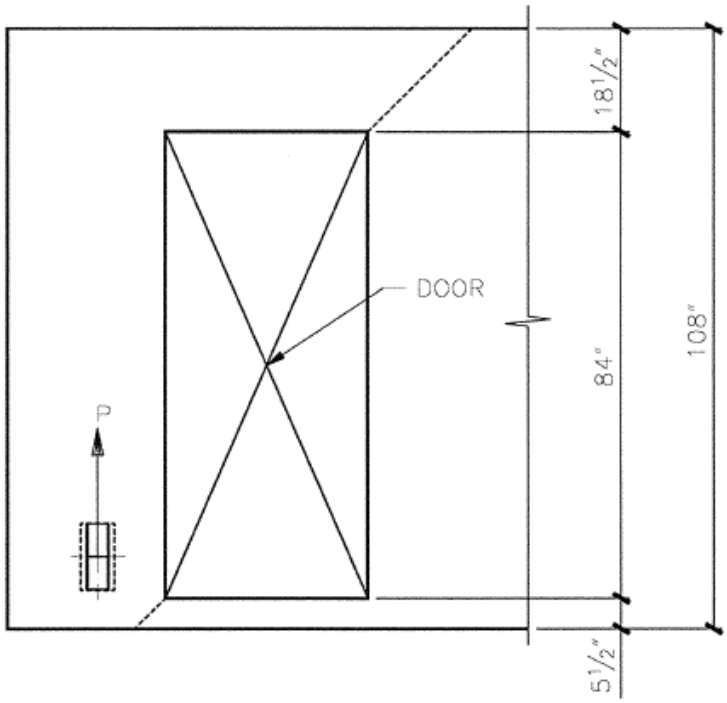


$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

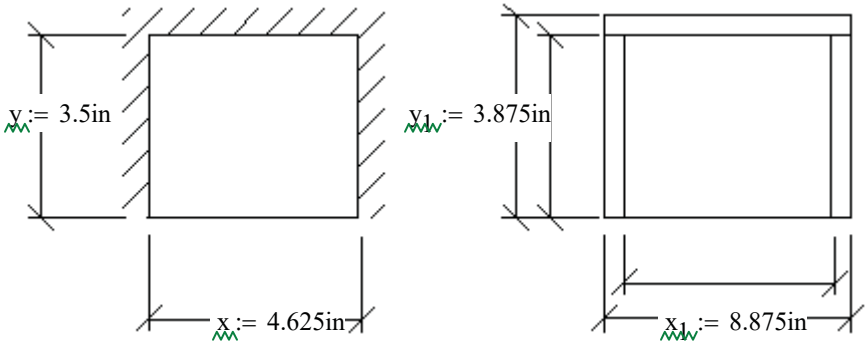
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

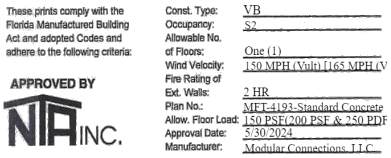
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 20.2 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 15.8 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2\max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 31.6 \cdot kip$ Wall Dead Load

$w_{ew} := 2\max(wt_{end})$ $w_{ew} = 10.2 \cdot kip$ Weight of End Wall

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 77.7 \cdot kip$ Factored Dead Load of Shelter

$$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}} \quad CG = 6.5\,ft$$ Center of gravity from bottom of shelter

Tie Down Bracket $Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 141.5 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}} \quad F_{turn} = 33.1 \cdot kip$$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 227.9 \cdot kip \cdot ft$ Overtruning Moment

$$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2} \quad M_{OT.Resist} = 297.4 \cdot kip \cdot ft$$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -69.5 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}} \quad F_{OT.trans} = -8.2 \cdot kip$$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans}\,DL$ $F_{net.slide.trans} = 17.6 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 8.8 \cdot kip$ Maximum Shear Force Per Tie-Down, θ_{min}

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 8.8 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$

$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right)$ $\phi P_{pl.FlrTie} = 26.928 \cdot kip$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.326$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.764$ < 1.0

$$\max\left(\frac{T_{tie.max}}{\phi P_{pl.FlrTie}}\right) + \frac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 1.09$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 1.09$ < 1.2

$chk_{8.Flr.Tie}$ = "OK"

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9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 21.22 \quad U_{R3.25} = 0.047$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2) \cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 4 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 1.8 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 26.22 \quad U_{R4.75} = 0.038$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (29.292 \quad 31.292 \quad 33.292 \quad 35.292)\,ft$ Length of Floor

$wgt_{flr}^T = (37.559 \quad 37.469 \quad 37.39 \quad 37.32 \quad NaN \quad NaN \quad NaN \quad NaN \quad NaN) \cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 3.82 \cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 4.18 \cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.719 \\ 0.68 \\ 16.095 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 18.494 \quad U_F = 0.054$

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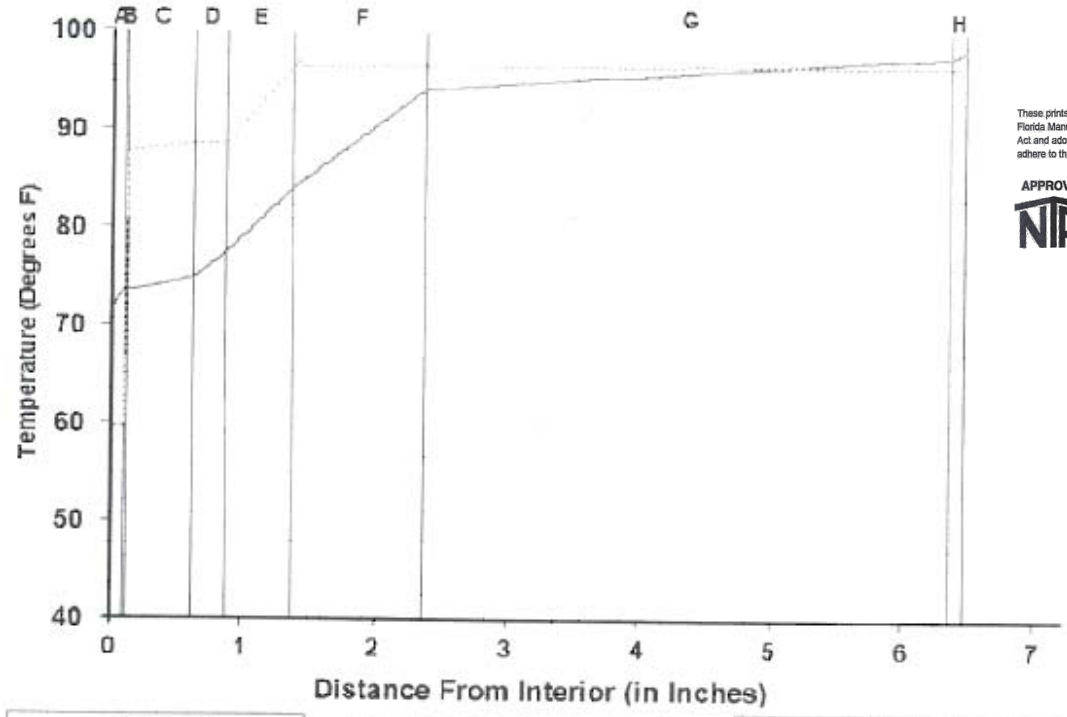
Dewpoint Analysis - Dow Chemical

Sam Cain



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Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

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10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

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Manufacturer:	Modular Connections, LLC

$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 65.753 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 935.2 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 1987 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$ Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.0101$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.242 \cdot in^2$ Req'd Area of Steel per foot Width

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 351 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG")$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 7.516\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 15.032\,ft$

Maximum allowable opening width

$w_{d.max} := \max(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4})$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

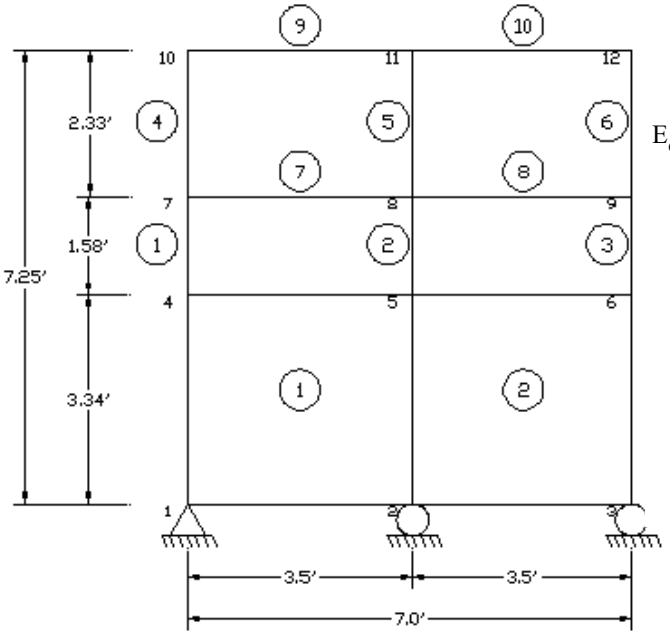
$chk_{opn.width} := if(w_d \geq w_{d.max}, "OK", "NG")$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.508 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.508 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.508 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2})$ In-Plane Force at Endwall



$V_u = 4.508 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in^2)	I (in^4)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

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$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.508 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.523 \cdot kip \cdot ft$

Shear

$\phi Vc_{6d.opn} := \phi_V \cdot 0.85 \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot \left(d_7 - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00115$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.145 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$ Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PSF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

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Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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APPENDIX "B"

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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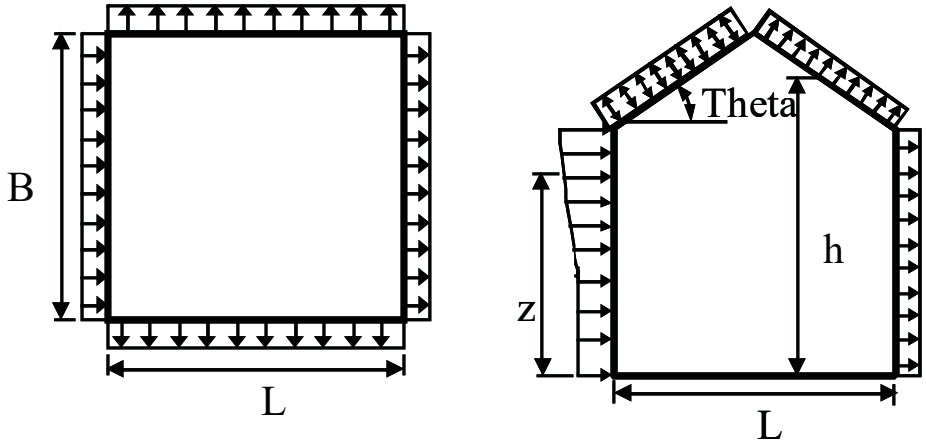
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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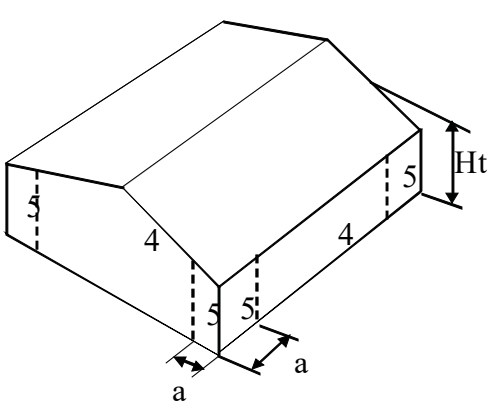
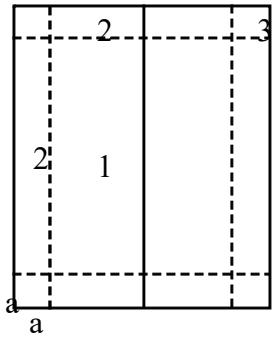
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Email: info@modularconnections.com

a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPENDIX "C"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

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MODULAR CONNECTIONS, LLC

Cover =

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

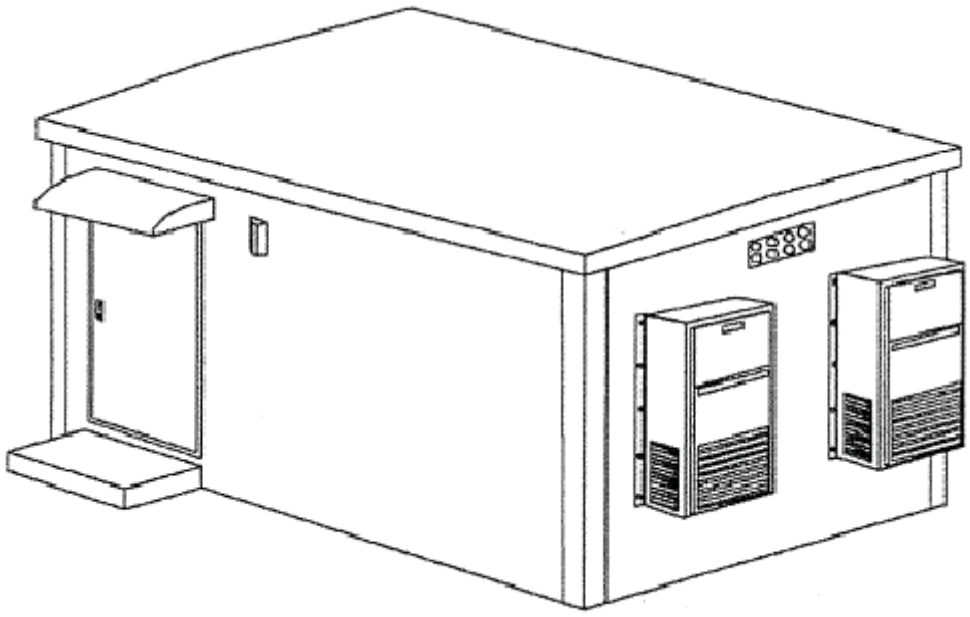
" IBC 2021 (ASCE 7-16) "

" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #4"

" REVISION - 0: DATE MARCH 11, 2024 "

""



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" copied or traced in whole or part without our written consent. "

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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" FLOOR DESIGN ---------- S4.1 - S4.31 "
" LATERAL DESIGN ---------- S5.1 - S5.24 "
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" CONNECTION DESIGN ---------- S7.1 - S7.33 "
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" FIRE RATING & THERMAL EFFICIENCY---------- S9.1 - S9.3 "
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" APPENDIX A - BULLET RESISTANCE TEST "
" APPENDIX B - AWINING / HOOD ATTACHMENT DESIGN "
" APPENDIX C - DEVELOPMENT OF REINFORCEMENT "

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER

2024.05.29 15:35:38-04'00'

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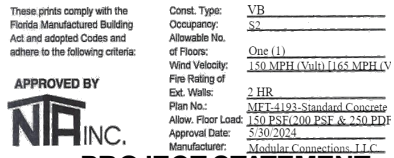
Email: info@modularconnections.com



PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 39.2 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 66.4 psf"
"Roof Live Load ---------- 100 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 150 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 35.8 psf"
"Uplift Wind Pressure, MWFRS ---------- 53.5 psf"
"Zone 4 Wind Pressure, C & C ---------- 53.3 psf"
"Zone 5 Wind Pressure, C & C ---------- 65.8 psf"
"Zone 1 Wind Pressure, C & C ---------- 78.2 psf"
"Zone 2 Wind Pressure, C & C ---------- 103.2 psf"
"Zone 3 Wind Pressure, C & C ---------- 140.7 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 32 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 39.5 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.73"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.18"

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$)
Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)
Welding electrodes : E70XX

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"
$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"
$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"
$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 100 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"
$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot psf \qquad q_h = 41.616 \cdot psf \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 20.81 & 35.79 \\ -25.18 & -10.2 \\ -32.25 & -17.27 \\ -53.48 & -38.49 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 35.79 & 20.81 \\ -10.2 & -25.18 \\ -17.27 & -32.25 \\ -38.49 & -53.48 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 35.79 \\ 25.18 \\ 32.25 \\ -53.48 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 45.99 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 41.616 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -65.8 & -50.8 \\ 5 & 20 \\ -78.2 & -63.3 \\ 5 & 20 \\ -103.2 & -88.2 \\ 5 & 20 \\ -140.7 & -125.7 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -50.8 & -65.8 \\ 20 & 5 \\ -63.3 & -78.2 \\ 20 & 5 \\ -88.2 & -103.2 \\ 20 & 5 \\ -125.7 & -140.7 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 53.3 \\ 53.3 \\ 65.8 \\ 78.2 \\ 103.2 \\ 140.7 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 65.8 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 78.2 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 103.2 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 140.7 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 107.369 \cdot psf$$

Max Roof Uplift Pressure (average)

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 1 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1.1 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 1.1$ $S_{MS.ref} =$ "Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.733$ $S_{DS.ref} =$ "5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} =$ "Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} =$ "5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"

$$H_E = \begin{pmatrix} 9.646 \\ 10.646 \\ 11.313 \end{pmatrix} \text{ft}$$

$H_{E.ref} =$ "Exterior Height of Building"

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right]$$

$Ta = 0.104$ $Ta_{ref} =$ "Approx Per., Eqn (12.8-7)"

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_0 = 0.229$ $T_s := \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_s = 1.144$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.1833$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E}\right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E}\right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1}\right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0323$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0\right)\right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max}\right), C_{s.min}\right)$ $C_s = 0.183$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 6 \cdot in$ Thickness @ ridge $t_{r.r} = 7.5 \cdot in$ $t_{r.a} = 6.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.162 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 66.38 \cdot psf$ $DL_{r.mn.ref}$:= "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 73.38 \cdot psf$ $DL_{r.mx.ref}$:= "Max. Roof DL"

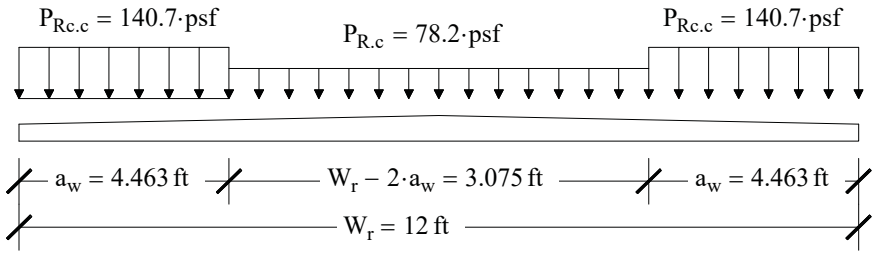
$w_{llr} = 100 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 140.7 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 78.2 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.463\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -24.358 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -112.769 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 10.762 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -10.762 \cdot psf$ Minimum Upward Roof Seismic Load

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$$LRFD = \begin{pmatrix} \text{"1a"} & \text{"1.4(D+F)"} \\ \text{"2a"} & \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"3a"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"3b+"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"3b-"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"4a+"} & \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"4a-"} & \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"5a+"} & \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"5a-"} & \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"6a+"} & \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"6a-"} & \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"7a+"} & \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"7a-"} & \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Wu_{S2.0} = \begin{pmatrix} 102.72 \\ 138.05 \\ 248.05 \\ 248.05 \\ 191.67 \\ 138.05 \\ 81.67 \\ 105.16 \\ 83.64 \\ 59.74 \\ -53.03 \\ 70.5 \\ 48.98 \end{pmatrix} \cdot psf$$

Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref} = \text{"Max Span of Roof Panel"}$

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 248.05 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -53.031 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

	$Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$	$Vu_{ref} = \text{"Ultimate Shear"}$
Roof Live	$Mu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}$	$Vu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}$
Load	$Mu_{2.1.mx} = 45.021 \cdot kip \cdot in$	$Vu_{2.1.mx} = 1.364\,kip$
Option 1	$Mu_{2.1.mn} := \min\left(0kip \cdot in, \lvert w_{u.mn} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$	$Vu_{2.1.mn} := \min\left(0kip, \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}\right)$
	$Mu_{2.1.mn} = 0 \cdot kip \cdot in$	$Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$

$B_{2.1} = \text{"#4"}$ $B_{2.1.ref} = \text{"Transverse Rebar Size"}$ $S_{2.1} = 10 \cdot in$ $S_{2.1.ref} = \text{"Transverse Rebar Spacing"}$

$B_{2.2} = \text{"#4"}$ $B_{2.2.ref} = \text{"Longitudinal Rebar Size"}$ $S_{2.2} = 12 \cdot in$ $S_{2.2.ref} = \text{"Longitudinal Rebar Spacing"}$

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref} = \text{"Clear Cover, From Bot. w/ Keyway"}$

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref} = \text{"Clear Cover, From Bot. w/o Keyway"}$

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref} = \text{"Roof Width Bar Diam."}$ $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref} = \text{"Roof Width Bar Area."}$

$As_{2.1} = 0.24 \cdot in^2$ $As_{2.1.ref} = \text{"Roof Width Area Reinf."}$ $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref} = \text{"Roof Length Bar Diam."}$

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref} = \text{"Roof Length Bar Area."}$ $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref} = \text{"Roof Length Area Reinf."}$

$A_{s.mim.2} = 0.162 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.24 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.004$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.314 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 69.7 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.38 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.03645$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 62.8 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 45 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 3.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 4.345 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 17.106 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 4.345 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 3.258 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 1.364 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{F.2.3} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\quad \rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $\quad a_{2.5} = 0.314 \cdot in$ $\quad a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $\quad M_{n.2.3} = 22.9 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $\quad c_{2.3} = 0.38 \cdot in$ $\quad c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\quad \varepsilon_{2.3} = 0.01081$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\quad \phi M_{n.2.3} = 20.6 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $\quad Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.5} = \text{"OK"}$

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{V.2.4} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\quad \lambda_{s.2.4} = 1$ $\quad \lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$\rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.a} = 2.158 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\quad \phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $\quad Vc_{2.4} = 2.158 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\quad \phi Vc_{2.4} = 1.618 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Vu_{2.1.mn} = 0 \cdot kip$ $\quad Vu_{ref} = \text{"Ultimate Shear"}$

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.6} = \text{"OK"}$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 307.547 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 3.375 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 38.97 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 31.468 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 13.318 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 153.773 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -10.544 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 76.887 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 76.89 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 307.5 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 157.3 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.02628 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.12139 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.09511 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.269\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.062\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.331\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 398.6$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

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Approval Date:	5/30/2024
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3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 100 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 100 \cdot psf$

$P_{W.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_0}$ = " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_1}$ = " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 65.8 \cdot psf$ $GC_{p.cc.max.ref_2}$ = "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 53.27 \\ 65.75 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 100 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.19 \\ 1.32 \\ 1.98 \\ 1.98 \\ 2.3 \\ 1.32 \\ 1.64 \\ 1.12 \\ 0.99 \\ 0.68 \\ 1.32 \\ 0.74 \\ 0.61 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 26.6 \\ 26.6 \\ 53.3 \\ 26.6 \\ 8.3 \\ -8.3 \\ 53.3 \\ 53.3 \\ 8.3 \\ -8.3 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 2.3 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.54 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.61 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.08 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.47 \cdot kip$

$P_{u.3b.h1}^T = (1.188 \;\; 1.318 \;\; 1.978 \;\; 1.978 \;\; 2.3 \;\; 1.318 \;\; 1.64 \;\; 1.121 \;\; 0.991 \;\; 0.677 \;\; 1.321 \;\; 0.742 \;\; 0.612) \cdot kip$

$M_{u.3b.h1}^T = (0 \;\; 0 \;\; 0 \;\; 0.27 \;\; 0.27 \;\; 0.539 \;\; 0.27 \;\; 0.084 \;\; -0.084 \;\; 0.539 \;\; 0.539 \;\; 0.084 \;\; -0.084) \cdot kip \cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.12 \;\; 0.12 \;\; 0.24 \;\; 0.12 \;\; 0.037 \;\; -0.037 \;\; 0.24 \;\; 0.24 \;\; 0.037 \;\; -0.037)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ & Roof Live Load $w_{llr} = 100\cdot psf$

$Pu_{3b.h2}^{T} = (1.25 \;\; 1.37 \;\; 2.03 \;\; 2.03 \;\; 2.35 \;\; 1.37 \;\; 1.69 \;\; 1.17 \;\; 1.05 \;\; 0.71 \;\; 1.36 \;\; 0.78 \;\; 0.65)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 8.3 \;\; -8.3 \;\; 53.3 \;\; 53.3 \;\; 8.3 \;\; -8.3)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2}) \qquad P_{u.3b.h2.mx} = 2.35\cdot kip \qquad M_{u.3b.h2.mx} := \max(M_{u.3b.h2}) \qquad M_{u.3b.h2.mx} = 0.67\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2}) \qquad P_{u.3b.h2.mn} = 0.65\cdot kip \qquad M_{u.3b.h2.mn} := \min(M_{u.3b.h2}) \qquad M_{u.3b.h2.mn} = -0.1\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|) \qquad V_{u.3b.h2.mx} = 0.52\cdot kip$

$P_{u.3b.h2}^{T} = (1.251 \;\; 1.372 \;\; 2.032 \;\; 2.032 \;\; 2.354 \;\; 1.372 \;\; 1.694 \;\; 1.175 \;\; 1.046 \;\; 0.712 \;\; 1.357 \;\; 0.777 \;\; 0.648)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.333 \;\; 0.333 \;\; 0.666 \;\; 0.333 \;\; 0.104 \;\; -0.104 \;\; 0.666 \;\; 0.666 \;\; 0.104 \;\; -0.104)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.133 \;\; 0.133 \;\; 0.266 \;\; 0.133 \;\; 0.042 \;\; -0.042 \;\; 0.266 \;\; 0.266 \;\; 0.042 \;\; -0.042)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667\,ft$ & Roof Live Load $w_{llr} = 100\cdot psf$

$Pu_{3b.h3}^{T} = (1.29 \;\; 1.41 \;\; 2.07 \;\; 2.07 \;\; 2.39 \;\; 1.41 \;\; 1.73 \;\; 1.21 \;\; 1.08 \;\; 0.74 \;\; 1.38 \;\; 0.8 \;\; 0.67)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 8.3 \;\; -8.3 \;\; 53.3 \;\; 53.3 \;\; 8.3 \;\; -8.3)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3}) \qquad P_{u.3b.h3.mx} = 2.39\cdot kip \qquad M_{u.3b.h3.mx} := \max(M_{u.3b.h3}) \qquad M_{u.3b.h3.mx} = 0.76\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3}) \qquad P_{u.3b.h3.mn} = 0.67\cdot kip \qquad M_{u.3b.h3.mn} := \min(M_{u.3b.h3}) \qquad M_{u.3b.h3.mn} = -0.12\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|) \qquad V_{u.3b.h3.mx} = 0.56\cdot kip$

$P_{u.3b.h3}^{T} = (1.293 \;\; 1.409 \;\; 2.069 \;\; 2.069 \;\; 2.391 \;\; 1.409 \;\; 1.731 \;\; 1.211 \;\; 1.082 \;\; 0.736 \;\; 1.38 \;\; 0.801 \;\; 0.671)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.379 \;\; 0.379 \;\; 0.758 \;\; 0.379 \;\; 0.118 \;\; -0.118 \;\; 0.758 \;\; 0.758 \;\; 0.118 \;\; -0.118)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.142 \;\; 0.142 \;\; 0.284 \;\; 0.142 \;\; 0.044 \;\; -0.044 \;\; 0.284 \;\; 0.284 \;\; 0.044 \;\; -0.044)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\qquad S_{3.0}$ = "N/A" $\cdot in \qquad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \qquad d_{3b} := \frac{t_w}{2} - 0.25in \qquad d_{3b} = 1.75\cdot in$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.557 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.087$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.92$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

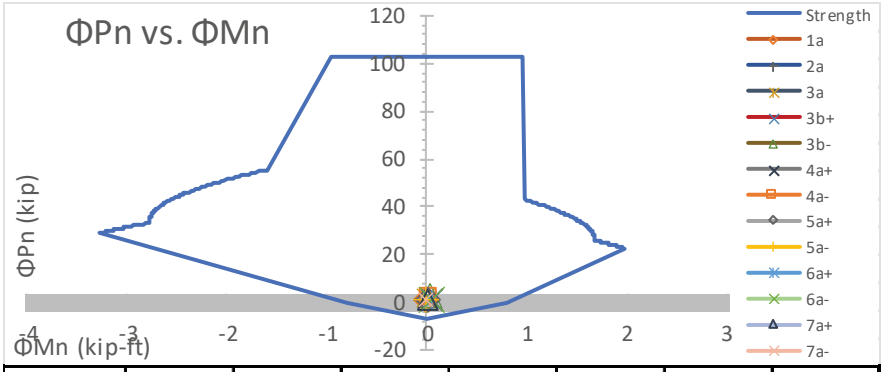
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.112$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.899$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.132$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.883$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.883$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.3%	1.4%	2.0%	2.94%	2.9%	6.1%	3.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.2%	1.1%	6.4%	6.1%	1.0%	1.0%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

MODULAR CONNECTIONS, LLC.
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Email: info@modularconnections.com

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref}$ = "Min. Space to Adjacent Opening"

$b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref}$ = "Strip Width Btwn Openings"

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1}$ = "#6" $B_{3.1.ref}$ = "Wall Vert. Perim. Reinf." $B_{3.2}$ = "#4" $B_{3.2.ref}$ = "Wall Top. Perim. Reinf."

$N_{B.3.1} = 1$ $N_{B.3.1.ref}$ = "# Wall Vert. Perim. Bars" $N_{B.3.2} = 2$ $N_{B.3.2.ref}$ = "# Wall Top Perim. Bars"

$D_{3c.1} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.2} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.1} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.2} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.3}$ = "#6" $B_{3.3.ref}$ = "EW Bot Perim. Reinf." $B_{3.4}$ = "#4" $B_{3.4.ref}$ = "SW Bot Perim. Reinf."

$N_{B.3.3} = 1$ $N_{B.3.3.ref}$ = "# EW Bot Perim. Bars" $N_{B.3.4} = 2$ $N_{B.3.4.ref}$ = "# SW Bot Perim. Bars"

$D_{3c.3} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.4} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.3} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.4} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.5}$ = "#5" $B_{3.5.ref}$ = "Wall Opening Reinf." $D_{3c.5} = 0.625 \cdot in$ D_{ref} = "Bar Diam."

$N_{B.3.5} = 2$ $N_{B.3.5.ref}$ = "# Wall Opening Bars" $A_{3c.5} = 0.62 \cdot in^2$ A_{ref} = "Area of Reinf."

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 100 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 33.2 \;\; -33.2 \;\; 213.1 \;\; 213.1 \;\; 33.2 \;\; -33.2) \cdot plf$

$P_{u.3c.h1d1}{}^T = (4.75 \;\; 5.27 \;\; 7.91 \;\; 7.91 \;\; 9.2 \;\; 5.27 \;\; 6.56 \;\; 4.48 \;\; 3.97 \;\; 2.71 \;\; 5.28 \;\; 2.97 \;\; 2.45) \cdot kip$

$M_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.2 \;\; -0.2 \;\; 1.31 \;\; 1.31 \;\; 0.2 \;\; -0.2) \cdot kip \cdot ft$

$V_{u.3c.h1d1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.12 \;\; -0.12 \;\; 0.75 \;\; 0.75 \;\; 0.12 \;\; -0.12) \cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d1.mx} = 9.2 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.31 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.46 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.45 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.2 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 100 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$W_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 24.9 \;\; -24.9 \;\; 159.8 \;\; 159.8 \;\; 24.9 \;\; -24.9)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.56 \;\; 3.95 \;\; 5.93 \;\; 5.93 \;\; 6.9 \;\; 3.95 \;\; 4.92 \;\; 3.36 \;\; 2.97 \;\; 2.03 \;\; 3.96 \;\; 2.22 \;\; 1.84)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.15 \;\; -0.15 \;\; 0.98 \;\; 0.98 \;\; 0.15 \;\; -0.15)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.09 \;\; -0.09 \;\; 0.56 \;\; 0.56 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 6.9\cdot kip$ $M_{u.3c.h1d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.1\cdot kip$

$P_{u.3c.h1d2.mn} = 1.84\cdot kip$ $M_{u.3c.h1d2.mn} = -0.15\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 20.8 \;\; -20.8 \;\; 133.2 \;\; 133.2 \;\; 20.8 \;\; -20.8)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.97 \;\; 3.29 \;\; 4.94 \;\; 4.94 \;\; 5.75 \;\; 3.29 \;\; 4.1 \;\; 2.8 \;\; 2.48 \;\; 1.69 \;\; 3.3 \;\; 1.85 \;\; 1.53)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.13 \;\; -0.13 \;\; 0.82 \;\; 0.82 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.07 \;\; -0.07 \;\; 0.47 \;\; 0.47 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 5.75\cdot kip$ $M_{u.3c.h1d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 0.91\cdot kip$

$P_{u.3c.h1d3.mn} = 1.53\cdot kip$ $M_{u.3c.h1d3.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 33.2 \;\; -33.2 \;\; 213.1 \;\; 213.1 \;\; 33.2 \;\; -33.2)\cdot plf$

$P_{u.3c.h2d1}^{T} = (5 \;\; 5.49 \;\; 8.13 \;\; 8.13 \;\; 9.42 \;\; 5.49 \;\; 6.78 \;\; 4.7 \;\; 4.18 \;\; 2.85 \;\; 5.43 \;\; 3.11 \;\; 2.59)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.2 \;\; -0.2 \;\; 1.31 \;\; 1.31 \;\; 0.2 \;\; -0.2)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.12 \;\; -0.12 \;\; 0.75 \;\; 0.75 \;\; 0.12 \;\; -0.12)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 9.42\cdot kip$ $M_{u.3c.h2d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.46\cdot kip$

$P_{u.3c.h2d1.mn} = 2.59\cdot kip$ $M_{u.3c.h2d1.mn} = -0.2\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 24.9 \;\; -24.9 \;\; 159.8 \;\; 159.8 \;\; 24.9 \;\; -24.9)\cdot plf$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.75 \;\; 4.12 \;\; 6.1 \;\; 6.1 \;\; 7.06 \;\; 4.12 \;\; 5.08 \;\; 3.52 \;\; 3.14 \;\; 2.14 \;\; 4.07 \;\; 2.33 \;\; 1.94)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.15 \;\; -0.15 \;\; 0.98 \;\; 0.98 \;\; 0.15 \;\; -0.15)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.09 \;\; -0.09 \;\; 0.56 \;\; 0.56 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 7.06\cdot kip$ $M_{u.3c.h2d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.1\cdot kip$

$P_{u.3c.h2d2.mn} = 1.94\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.15\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 20.8 \;\; -20.8 \;\; 133.2 \;\; 133.2 \;\; 20.8 \;\; -20.8)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (3.13 \;\; 3.43 \;\; 5.08 \;\; 5.08 \;\; 5.89 \;\; 3.43 \;\; 4.24 \;\; 2.94 \;\; 2.61 \;\; 1.78 \;\; 3.39 \;\; 1.94 \;\; 1.62)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.13 \;\; -0.13 \;\; 0.82 \;\; 0.82 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.07 \;\; -0.07 \;\; 0.47 \;\; 0.47 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 5.89\cdot kip$ $M_{u.3c.h2d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 0.91\cdot kip$

$P_{u.3c.h2d3.mn} = 1.62\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.13\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 33.2 \;\; -33.2 \;\; 213.1 \;\; 213.1 \;\; 33.2 \;\; -33.2)\cdot plf$

$P_{u.3c.h3d1}^{T} = (5.17 \;\; 5.63 \;\; 8.27 \;\; 8.27 \;\; 9.56 \;\; 5.63 \;\; 6.92 \;\; 4.84 \;\; 4.33 \;\; 2.94 \;\; 5.52 \;\; 3.2 \;\; 2.69)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.2 \;\; -0.2 \;\; 1.31 \;\; 1.31 \;\; 0.2 \;\; -0.2)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.12 \;\; -0.12 \;\; 0.75 \;\; 0.75 \;\; 0.12 \;\; -0.12)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 9.56\cdot kip$ $M_{u.3c.h3d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.46\cdot kip$

$P_{u.3c.h3d1.mn} = 2.69\cdot kip$ $M_{u.3c.h3d1.mn} = -0.2\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67\,ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 24.9 \;\; -24.9 \;\; 159.8 \;\; 159.8 \;\; 24.9 \;\; -24.9)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.88 \;\; 4.23 \;\; 6.21 \;\; 6.21 \;\; 7.17 \;\; 4.23 \;\; 5.19 \;\; 3.63 \;\; 3.25 \;\; 2.21 \;\; 4.14 \;\; 2.4 \;\; 2.01)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.15 \;\; -0.15 \;\; 0.98 \;\; 0.98 \;\; 0.15 \;\; -0.15)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.09 \;\; -0.09 \;\; 0.56 \;\; 0.56 \;\; 0.09 \;\; -0.09)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 7.17\cdot kip$ $M_{u.3c.h3d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.1\cdot kip$

$P_{u.3c.h3d2.mn} = 2.01\cdot kip$ $M_{u.3c.h3d2.mn} = -0.15\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67 ft$ Roof Live Load $w_{llr} = 100\cdot psf$ & Door Width Less <= $w_{door_2} = 3 ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 20.8 \;\; -20.8 \;\; 133.2 \;\; 133.2 \;\; 20.8 \;\; -20.8)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.13 \;\; -0.13 \;\; 0.82 \;\; 0.82 \;\; 0.13 \;\; -0.13)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.07 \;\; -0.07 \;\; 0.47 \;\; 0.47 \;\; 0.07 \;\; -0.07)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 5.98\cdot kip$ $M_{u.3c.h3d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 0.91\cdot kip$

$P_{u.3c.h3d3.mn} = 1.68\cdot kip$ $M_{u.3c.h3d3.mn} = -0.13\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - (0.75in + 0.5D_{3c.5})\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.463\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.097\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 0.914\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{(k_{wall}\cdot H_{wall})^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

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$\delta_{3c.h1d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.469$ $\Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d1} = 0.681$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

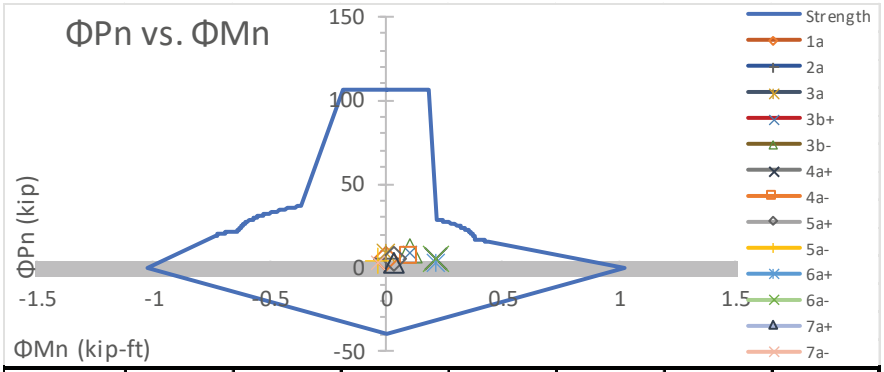
$\delta_{3c.h2d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d1} = 1.677$ $\Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1}$ $\Omega_{\delta.3c.h2d1} = 0.596$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h3d1} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d1} = 3.326$ $\Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1}$ $\Omega_{\delta.3c.h3d1} = 0.301$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3c.d1} := \min(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1})$ $\Omega_{\delta.3c.d1} = 0.301$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.9%	5.3%	7.8%	15.17%	16.3%	26.5%	14.1%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.5%	4.1%	23.4%	26.3%	3.7%	3.4%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$\delta_{3c.h1d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}}$ $\delta_{3c.h1d1} = 1.469$ $\Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1}$ $\Omega_{\delta.3c.h1d2} = 0.681$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h2d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}}$ $\delta_{3c.h2d2} = 1.434$ $\Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1}$ $\Omega_{\delta.3c.h2d2} = 0.697$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\delta_{3c.h3d2} := \dfrac{1}{1 - \dfrac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}}$ $\delta_{3c.h3d2} = 2.103$ $\Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1}$ $\Omega_{\delta.3c.h3d2} = 0.476$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$\Omega_{\delta.3c.d2} := \min(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2})$ $\Omega_{\delta.3c.d2} = 0.476$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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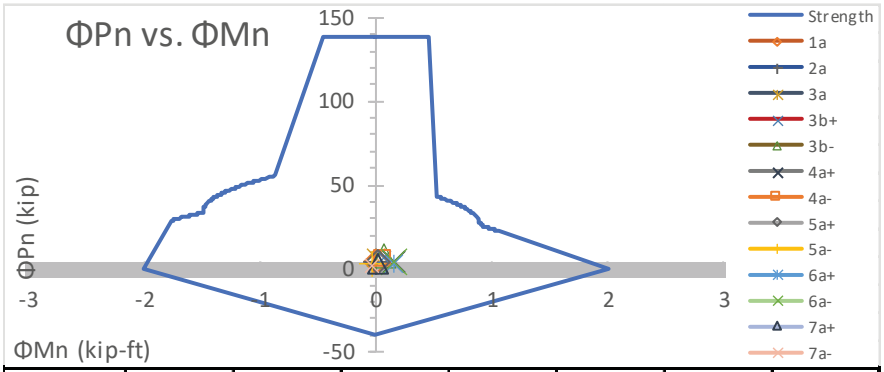
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.8%	3.1%	4.5%	4.59%	5.2%	8.8%	4.5%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.6%	2.3%	8.4%	8.7%	1.7%	1.5%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.249 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.8$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

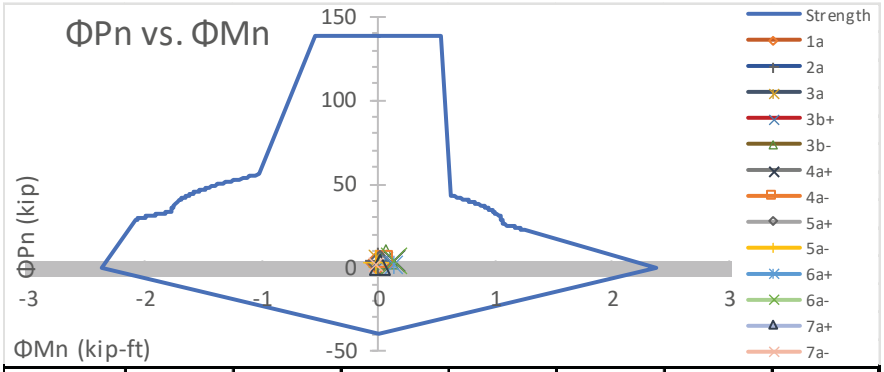
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.337 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.748$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.776 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.563$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.563$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.3%	2.5%	3.7%	3.74%	4.3%	6.1%	3.1%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.2%	2.0%	5.8%	6.1%	1.4%	1.2%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

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d. Wall - Header Check (Above Opening) $w_{llr} = 100 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in$ Min Header Height for Opening Widths of $w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$Wu_{3d.0}^{T} = (827.9 \quad 1009.6 \quad 1669.6 \quad 1669.6 \quad 1991.7 \quad 1009.6 \quad 1331.7 \quad 812.3 \quad 683.2 \quad 476.4 \quad 1120.6 \quad 541 \quad 411.9) \cdot plf$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1991.7 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in$ $A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for for:
$w_{llr} = 100 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min\left(Vc_{3d.0.a}, Vc_{3d.0.b}\right)$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot \left(w_{open_0}\right)^2}{8}$ $Mu_{3d.0} = 8.96 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max\left(\left|wu_{3d.0mx}\right|, \left|wu_{3d.0mn}\right|\right) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 5.98 \cdot kip$

$chk_{3d.1} := \text{if}\left(\phi Mn_{3d.0} \geq Mu_{3d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot \left[f_y \cdot \left(0.5 \cdot h_{3d_0}\right)\right]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := \text{if}\left(chk_{3d.2} = \text{"OK"}, \text{"OK"}, \text{if}\left(chk_{3d.2} = \text{"Stirrups Req'd"}, \text{if}\left(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, \text{"NG"}, \text{"OK"}\right), \text{"OK"}\right)\right)$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (727.4 \quad 923.5 \quad 1583.5 \quad 1583.5 \quad 1905.6 \quad 923.5 \quad 1245.6 \quad 726.2 \quad 597 \quad 420.4 \quad 1064.6 \quad 484.9 \quad 355.8) \cdot plf$

$wu_{3d.1mx} := \max\left(Wu_{3d.1}\right)$ $wu_{3d.1mx} = 1905.6 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min\left(Wu_{3d.1}, 0plf\right)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 100 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}\left(a_{3d.1.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots + A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := \text{root}\left(f_{3d.1}\left(a_{3d.1.g}\right), a_{3d.1.g}\right)$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$F2_{3d.1} := A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right]$ $F2_{3d.1} = 11.7 \cdot kip$

$F3_{3d.1} := A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right]$ $F3_{3d.1} = 7.2 \cdot kip$

$F4_{3d.1} := A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$ $F4_{3d.1} = 18.6 \cdot kip$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772 \cdot kip$

$Mn_{3d.1} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} \cdot \frac{a_{3d.1}}{2} + F1_{3d.1} \cdot d_{3d.1_0} + F2_{3d.1} \cdot d_{3d.1_1} \ldots + F3_{3d.1} \cdot d_{3d.1_2} + F4_{3d.1} \cdot d_{3d.1_3}$ $Mn_{3d.1} = 38.5 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}} \cdot d_{3d.1_3}$ $\varepsilon_{3d.1} = 0.014$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{3d.1} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.1} := \phi_{3d.1} \cdot Mn_{3d.1}$ $\phi Mn_{3d.1} = 34.64 \cdot k \cdot ft$ $Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375 \cdot in$

$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) \cdot t_w}$ $\rho_{3d.1} = 0.0062$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.1} := \sum \begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.1_0}\right)}\right]$ $\rho'_{3d.1} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10 \cdot in}}}\right)$ $\lambda_{s.3d.1} = 0.876$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.1.a} := 8 \cdot \lambda_{s.3d.1} \cdot \lambda \cdot \rho_{3d.1}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.a} = 5.48 \cdot kip$

$Vc_{3d.1.b} = 21.31 \cdot kip$

$\phi Vc_{3d.1} := \phi_V \cdot \min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right)$ $\phi Vc_{3d.1} = 4.1 \cdot kip$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.1} := \frac{wu_{3d.1mx} \cdot \left(w_{open_1}\right)^2}{8}$ $Mu_{3d.1} = 3.81 \cdot kip \cdot ft$ $Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right) \cdot w_{open_1}}{2}$ $Vu_{3d.1} = 3.81 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h3d_1\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 100 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (706.3 \quad 905.4 \quad 1565.4 \quad 1565.4 \quad 1887.5 \quad 905.4 \quad 1227.5 \quad 708 \quad 578.9 \quad 408.6 \quad 1052.8 \quad 473.1 \quad 344) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1887.5 \cdot plf$ Max. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h3d_2 - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h3d_2 - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 100 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 3.81 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 2.83 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2} \\ \text{"Min. Stirrups Req'd"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2} \\ \text{"Stirrups Req'd"} & \text{otherwise} \end{cases}$ $\quad chk_{3d.10} = \text{"Min. Stirrups Req'd"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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Email: info@modularconnections.com

4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
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Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 3.47 \cdot psf$		$M_{E.4a} = 0.001 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.003 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.235 \\ 0.235 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.081 \\ 0.081 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.193 \\ 0.193 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.063 \\ 0.063 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.152 \\ 0.152 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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Const. Type: VB
Occupancy: S2
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Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 10.71 \cdot plf$		$M_{E.4b} = 0.117 \cdot kip \cdot ft$	$V_{E.4b} = 0.05 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.51 \\ 6.51 \\ 0.58 \\ 0.58 \\ 0.69 \\ 0.69 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.79 \\ 2.79 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.43 \\ 5.43 \\ 0.58 \\ 0.58 \\ 0.69 \\ 0.69 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.33 \\ 2.33 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.34 \\ 4.34 \\ 0.58 \\ 0.58 \\ 0.69 \\ 0.69 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.86 \\ 1.86 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

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Occupancy: S2
Allowable No. of Floors: One (1)
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Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.co

$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Email: info@modularconnections.co

$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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Occupancy: S2
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Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (4 \quad 5 \quad 6 \quad 7 \quad 8 \quad 9 \quad 10 \quad 10 \quad 11)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (11.29 \quad 13.29 \quad 15.29 \quad 17.29 \quad 19.29 \quad 21.29 \quad 23.29 \quad 25.29 \quad 27.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (5.93 \quad 6.3 \quad 6.57 \quad 6.78 \quad 6.95 \quad 7.08 \quad 7.19 \quad 6.62 \quad 6.75) \cdot psf$ Floor Transverse Ribs

$wgt_{f.l.rib}^T = (3.31 \quad 3.38 \quad 3.42 \quad 3.46 \quad 3.49 \quad 3.51 \quad 3.53 \quad 3.61 \quad 3.62) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (8.49 \quad 7.21 \quad 6.27 \quad 5.54 \quad 4.97 \quad 4.5 \quad 4.12 \quad 3.79 \quad 3.51) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (3.74 \quad 3.84 \quad 3.91 \quad 3.96 \quad 4.01 \quad 4.04 \quad 4.07 \quad 4.09 \quad 4.11) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (41.1 \quad 40.4 \quad 39.8 \quad 39.4 \quad 39.1 \quad 38.8 \quad 38.6 \quad 37.8 \quad 37.7) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (45.1 \quad 44.4 \quad 43.8 \quad 43.4 \quad 43.1 \quad 42.8 \quad 42.6 \quad 41.8 \quad 41.7) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

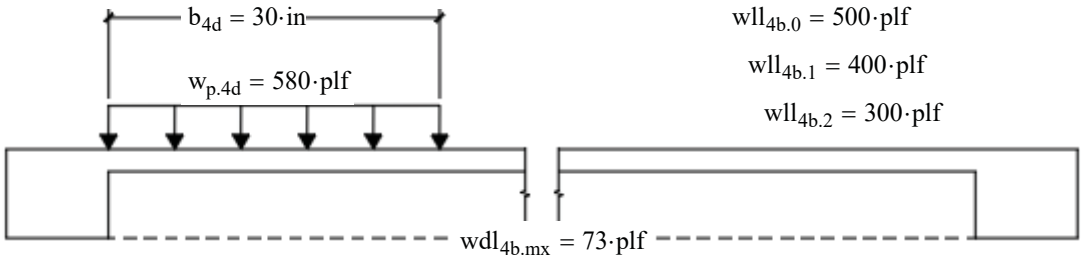
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

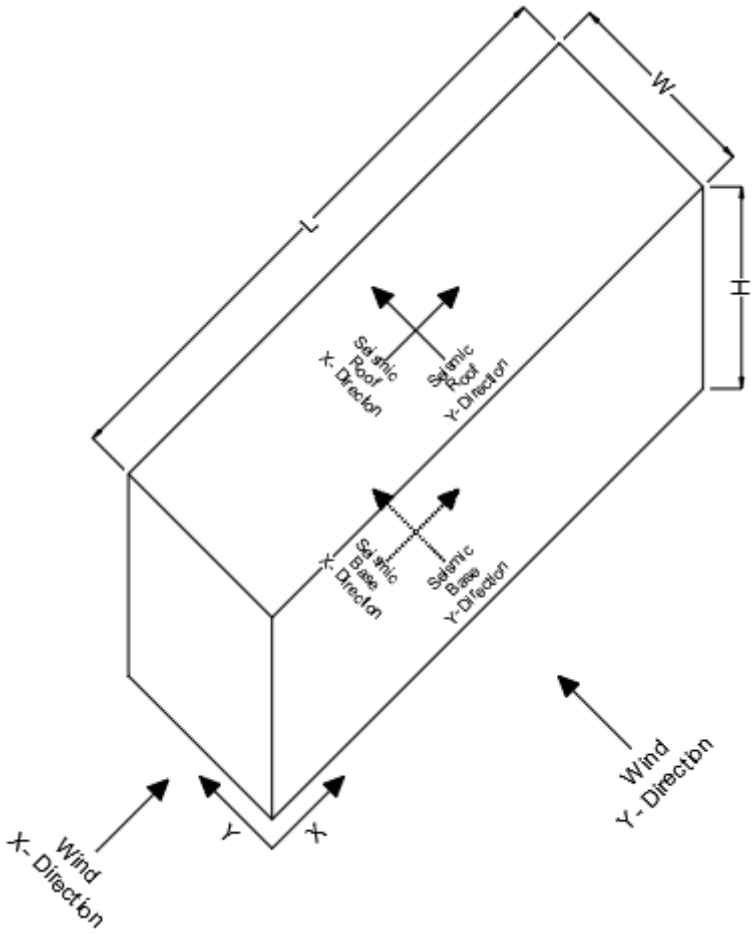
$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx} \cdot (L_r) \cdot (W_r)$

$Wt_{rf.mn} := DL_{r.mn} \cdot (L_r) \cdot (W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25 \cdot w_{llr}, 0psf)$

$w_{llr.eq} = 25 \cdot psf$

$Wt_{25ll} := w_{llr.eq} \cdot [(L_r) \cdot (W_r)]$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{rf.mx}^{T} = (10.9 \quad 12.6 \quad 14.4 \quad 16.1 \quad 17.9 \quad 19.7 \quad 21.4 \quad 23.2 \quad 24.9) \cdot kip$

Max Roof Weight, $w_{llr} = 100 \cdot psf$

$Wt_{25ll}^{T} = (3.7 \quad 4.3 \quad 4.9 \quad 5.5 \quad 6.1 \quad 6.7 \quad 7.3 \quad 7.9 \quad 8.5) \cdot kip$

25% Roof LLr 0, $w_{llr} = 100 \cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (15 \quad 17 \quad 19 \quad 22 \quad 24 \quad 26 \quad 29 \quad 31 \quad 33) \cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (9.8 \quad 11.4 \quad 13 \quad 14.6 \quad 16.2 \quad 17.8 \quad 19.4 \quad 21 \quad 22.6) \cdot kip$

Min Roof Weight, $w_{llr} = 100 \cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75 \cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3} \cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128) \cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (115.5 \quad 127.5 \quad 135.5) \cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 111.75 \\ 123.75 \\ 131.75 \end{pmatrix} \cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r} \cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r} \cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r} \cdot el_{avg.rf_2}$

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Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{w.h0.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{EQ.w.h0}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{EQ.w.h1}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{EQ.w.h2}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

$el_{top.w} := h_{wl\_fl} + H_{wall}$ $\quad el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$ Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean\left(h_{wl\_fl}, el_{top.w_0}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_1}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_2}\right) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$ $\quad W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$ $\quad W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if\left(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf\right)$ $\quad w_{llf.0.eq} = 62.5 \cdot psf$ $\quad Wt_{25llf0} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.1.eq} := if\left(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf\right)$ $\quad w_{llf.1.eq} = 50 \cdot psf$ $\quad Wt_{25llf1} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.2.eq} := if\left(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf\right)$ $\quad w_{llf.2.eq} = 37.5 \cdot psf$ $\quad Wt_{25llf2} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$ $\quad Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (5.6 \quad 6.5 \quad 7.3 \quad 8.2 \quad 9.1 \quad 10 \quad 10.9 \quad 11.6 \quad 12.5) \cdot kip$ Max Floor Weight

$Wt_{f.mn}^T = (5.1 \quad 5.9 \quad 6.7 \quad 7.5 \quad 8.3 \quad 9.1 \quad 9.8 \quad 10.5 \quad 11.3) \cdot kip$ Min Floor Weight

$Wt_{25llf0}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$ $\quad Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$ $\quad Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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$Wt_{EQ.f.f0}^{T} = (13.3 \;\; 15.6 \;\; 17.8 \;\; 20.1 \;\; 22.3 \;\; 24.6 \;\; 26.8 \;\; 28.9 \;\; 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (13.3 \;\; 15.6 \;\; 17.8 \;\; 20.1 \;\; 22.3 \;\; 24.6 \;\; 26.8 \;\; 28.9 \;\; 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (13.3 \;\; 15.6 \;\; 17.8 \;\; 20.1 \;\; 22.3 \;\; 24.6 \;\; 26.8 \;\; 28.9 \;\; 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 45.99\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (5 \;\; 5.9 \;\; 6.8 \;\; 7.7 \;\; 8.6 \;\; 9.4 \;\; 10.3 \;\; 11.2 \;\; 12.1)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (5.5 \;\; 6.5 \;\; 7.5 \;\; 8.5 \;\; 9.4 \;\; 10.4 \;\; 11.4 \;\; 12.4 \;\; 13.4)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (5.9 \;\; 6.9 \;\; 8 \;\; 9 \;\; 10 \;\; 11.1 \;\; 12.1 \;\; 13.2 \;\; 14.2)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 5.17\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 5.71\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 6.07\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (2.5 \;\; 2.9 \;\; 3.4 \;\; 3.8 \;\; 4.3 \;\; 4.7 \;\; 5.2 \;\; 5.6 \;\; 6.1)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (2.8 \;\; 3.3 \;\; 3.7 \;\; 4.2 \;\; 4.7 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.7)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (2.9 \;\; 3.5 \;\; 4 \;\; 4.5 \;\; 5 \;\; 5.5 \;\; 6.1 \;\; 6.6 \;\; 7.1)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 2.59\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 2.86\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 3.03\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.183$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{EQ.bh0f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh1f0} := C_s\cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f0}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh2f0} := C_s\cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh0f1}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh0f1} := C_s\cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f1}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh1f1} := C_s\cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f1}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh2f1} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh0f2}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh0f2} := C_s\cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f2}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh1f2} := C_s\cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f2}^{T} = (47 \;\; 53 \;\; 59 \;\; 65 \;\; 72 \;\; 78 \;\; 84 \;\; 90 \;\; 96)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 100\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh2f2} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 13 \;\; 14 \;\; 15 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.rh0}^{T} = (24 \;\; 27 \;\; 30 \;\; 33 \;\; 37 \;\; 40 \;\; 43 \;\; 46 \;\; 49)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 100\cdot psf$ $H_{wall_0} = 9\,ft$

$V_{EQ.rh0} := C_s\cdot Wt_{EQ.rh0}$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.co

$V_{EQ.rh0}^{T} = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

$Wt_{EQ.rh1}^{T} = (24 \;\; 27 \;\; 30 \;\; 33 \;\; 37 \;\; 40 \;\; 43 \;\; 46 \;\; 49)\cdot kip$

Roof Diap. Wt/Shear
$w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^{T} = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

$Wt_{EQ.rh2}^{T} = (24 \;\; 27 \;\; 30 \;\; 33 \;\; 37 \;\; 40 \;\; 43 \;\; 46 \;\; 49)\cdot kip$

Roof Diap. Wt/Shear
$w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.7\,ft$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^{T} = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r}$ $\quad w_{EQh0.y}^{T} = (356 \;\; 347 \;\; 340 \;\; 335 \;\; 330 \;\; 327 \;\; 324 \;\; 321 \;\; 319)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r}$ $\quad w_{EQh1.y}^{T} = (356 \;\; 347 \;\; 340 \;\; 335 \;\; 330 \;\; 327 \;\; 324 \;\; 321 \;\; 319)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r}$ $\quad w_{EQh2.y}^{T} = (356 \;\; 347 \;\; 340 \;\; 335 \;\; 330 \;\; 327 \;\; 324 \;\; 321 \;\; 319)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r}$ $\quad w_{EQh0.x}^{T} = (366 \;\; 414 \;\; 463 \;\; 511 \;\; 560 \;\; 608 \;\; 657 \;\; 705 \;\; 754)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r}$ $\quad w_{EQh1.x}^{T} = (366 \;\; 414 \;\; 463 \;\; 511 \;\; 560 \;\; 608 \;\; 657 \;\; 705 \;\; 754)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r}$ $\quad w_{EQh2.x}^{T} = (366 \;\; 414 \;\; 463 \;\; 511 \;\; 560 \;\; 608 \;\; 657 \;\; 705 \;\; 754)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 100\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r}$ $\quad w_{W.h0.y}^{T} = (203 \;\; 206 \;\; 208 \;\; 209 \;\; 210 \;\; 211 \;\; 212 \;\; 213 \;\; 214)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r}$ $\quad w_{W.h1.y}^{T} = (224 \;\; 227 \;\; 229 \;\; 231 \;\; 232 \;\; 233 \;\; 234 \;\; 235 \;\; 236)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r}$ $\quad w_{W.h2.y}^{T} = (238 \;\; 241 \;\; 244 \;\; 245 \;\; 247 \;\; 248 \;\; 249 \;\; 250 \;\; 251)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67\,ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$

$w_{W.h0.x} = 215.6\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$

$w_{W.h1.x} = 238\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$

$w_{W.h2.x} = 252.9\cdot plf$

R Diap. Wind Dist. Load
$H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$M_{u.bh0.y}^{T} = (6.8 \;\; 8.9 \;\; 11.3 \;\; 14.1 \;\; 17.1 \;\; 20.4 \;\; 24 \;\; 27.9 \;\; 32)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (6.8 \;\; 8.9 \;\; 11.3 \;\; 14.1 \;\; 17.1 \;\; 20.4 \;\; 24 \;\; 27.9 \;\; 32)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (6.8 \;\; 8.9 \;\; 11.3 \;\; 14.1 \;\; 17.1 \;\; 20.4 \;\; 24 \;\; 27.9 \;\; 32)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.bh0.x}^{T} = (6.6 \;\; 7.5 \;\; 8.3 \;\; 9.2 \;\; 10.1 \;\; 10.9 \;\; 11.8 \;\; 12.7 \;\; 13.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (6.6 \;\; 7.5 \;\; 8.3 \;\; 9.2 \;\; 10.1 \;\; 10.9 \;\; 11.8 \;\; 12.7 \;\; 13.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (6.6 \;\; 7.5 \;\; 8.3 \;\; 9.2 \;\; 10.1 \;\; 10.9 \;\; 11.8 \;\; 12.7 \;\; 13.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 100\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.b.h0.y}^{T} = (3.9 \;\; 5.3 \;\; 6.9 \;\; 8.8 \;\; 10.9 \;\; 13.2 \;\; 15.7 \;\; 18.5 \;\; 21.4)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (4.3 \;\; 5.8 \;\; 7.6 \;\; 9.7 \;\; 12 \;\; 14.5 \;\; 17.3 \;\; 20.4 \;\; 23.7)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (4.5 \;\; 6.2 \;\; 8.1 \;\; 10.3 \;\; 12.8 \;\; 15.5 \;\; 18.4 \;\; 21.7 \;\; 25.1)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{u.b.h0.y}^{T} = (1.3 \quad 1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.4 \quad 2.6 \quad 2.8 \quad 3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (1.4 \quad 1.6 \quad 1.9 \quad 2.1 \quad 2.4 \quad 2.6 \quad 2.9 \quad 3.1 \quad 3.3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (1.5 \quad 1.7 \quad 2 \quad 2.2 \quad 2.5 \quad 2.8 \quad 3 \quad 3.3 \quad 3.5)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 3.9\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 4.3\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 4.6\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

$v_{u.b.h0.x} = 1.3\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.4\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = "\#6"$ $B_{7.1.ref} = "Roof\ Perim.\ Reinf."$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = "\#\ Roof\ Perim.\ Bars"$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = "Roof\ Perim.\ Reiinf.\ Bar\ Diam."$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = "Roof\ Perim.\ Reinf.\ Area"$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := "Roof\ Diaph.Reinf."$ $\rho_{b.1} := \frac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00074$

$d_{b.1} := W_r - 0.75in - \frac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \frac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 1.67\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \frac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\frac{d_{b.1}}{2} - \frac{a_{b.1}}{2}\right)$ $M_{b.1} = 383.1\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \frac{a_{b.1}}{0.85}$ $c_{b.1} = 2\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\frac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.21474$ Strain in Ten. Rebar

$\phi_{b.1} := if\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, if\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 344.8\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\frac{M_{u.bh0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.bh1.y}}{\phi M_{b.1_1}} \quad \frac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 2 & 2.6 & 3.3 & 4.1 & 5 & 5.9 & 7 & 8.1 & 9.3 \\ 2 & 2.6 & 3.3 & 4.1 & 5 & 5.9 & 7 & 8.1 & 9.3 \\ 2 & 2.6 & 3.3 & 4.1 & 5 & 5.9 & 7 & 8.1 & 9.3 \end{pmatrix}\cdot\%$

$chk_{b.1a} := if\left(\max\left(Unity_{b.1a}\right) < 1, "OK", "NG"\right)$

$chk_{b.1a} = "OK"$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.1b}^T = \begin{pmatrix} 1.1 & 1.5 & 2 & 2.5 & 3.2 & 3.8 & 4.6 & 5.4 & 6.2 \\ 1.2 & 1.7 & 2.2 & 2.8 & 3.5 & 4.2 & 5 & 5.9 & 6.9 \\ 1.3 & 1.8 & 2.4 & 3 & 3.7 & 4.5 & 5.3 & 6.3 & 7.3 \end{pmatrix} \cdot \%$$

$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.68 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof\ Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 1.673 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$

$d_{b.2}^T = (12.26 \quad 14.26 \quad 16.26 \quad 18.26 \quad 20.26 \quad 22.26 \quad 24.26 \quad 26.26 \quad 28.26)\,ft$

$\rho_{b.2}^T = (0.00077 \quad 0.00066 \quad 0.00058 \quad 0.00052 \quad 0.00046 \quad 0.00042 \quad 0.00039 \quad 0.00036 \quad 0.00033)$

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.2}}{2} \right)$$

$M_{b.2a}^T = (409 \quad 476 \quad 543 \quad 611 \quad 678 \quad 745 \quad 812 \quad 879 \quad 947) \cdot kip \cdot ft$ Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 2.1 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$\varepsilon_{t.b.2a}^T = (0.208 \quad 0.2425 \quad 0.2769 \quad 0.3113 \quad 0.3457 \quad 0.3802 \quad 0.4146 \quad 0.449 \quad 0.4834)$ Strain in Rebar

$\phi_{b.2} := if\left[min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50 \right] \right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^T = (368 \quad 429 \quad 489 \quad 550 \quad 610 \quad 670 \quad 731 \quad 791 \quad 852) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.2a}^T = \begin{pmatrix} 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.2b}^T = \begin{pmatrix} 1.1 & 0.9 & 0.8 & 0.7 & 0.6 & 0.6 & 0.5 & 0.5 & 0.5 \\ 1.2 & 1 & 0.9 & 0.8 & 0.7 & 0.6 & 0.6 & 0.5 & 0.5 \\ 1.2 & 1.1 & 0.9 & 0.8 & 0.7 & 0.7 & 0.6 & 0.6 & 0.5 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 864 \cdot in^2$ Roof Shear Area for $t_{r.e} = 6 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00074$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e}\cdot L_{r_0} \\ t_{r.e}\cdot L_{r_1} \\ t_{r.e}\cdot L_{r_2} \\ t_{r.e}\cdot L_{r_3} \\ t_{r.e}\cdot L_{r_4} \\ t_{r.e}\cdot L_{r_5} \\ t_{r.e}\cdot L_{r_6} \\ t_{r.e}\cdot L_{r_7} \\ t_{r.e}\cdot L_{r_8} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 888 \\ 1032 \\ 1176 \\ 1320 \\ 1464 \\ 1608 \\ 1752 \\ 1896 \\ 2040 \end{pmatrix}\cdot in^2 \qquad \text{Roof Shear Area for } t_{r.e} = 6\cdot in \ \& \ L_r = \begin{pmatrix} 1776 \\ 2064 \\ 2352 \\ 2640 \\ 2928 \\ 3216 \\ 3504 \\ 3792 \\ 4080 \end{pmatrix}\cdot\frac{in^2}{ft}$$

$$\rho_{b.2}^{T} = (0.00077 \quad 0.00066 \quad 0.00058 \quad 0.00052 \quad 0.00046 \quad 0.00042 \quad 0.00039 \quad 0.00036 \quad 0.00033)$$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv}\cdot\left(2\cdot\sqrt{f'_c} + \rho\cdot f_y\right)$ $V_{b.3a.1.mx} = 154.3\cdot kip$ Max Allowable for $t_{r.e} = 6\cdot in$ & $W_r = 12\cdot ft$

$$V_{b.3a.2.mx}^{T} = (159.9 \quad 179.3 \quad 198.6 \quad 217.9 \quad 237.2 \quad 256.5 \quad 275.9 \quad 295.2 \quad 314.5)\cdot kip$$

Max Allowable Shear for $t_{r.e} = 6\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$V_{b.3b.1} = 49.3\cdot kip$ Nominal Shear Strength for $t_{r.e} = 6\cdot in$ & $W_r = 12\cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Roof Diaph. Shear Strength for $t_{r.e} = 6\cdot in$ & $W_r = 12\cdot ft$ $\phi V_{c.b.3a.1} = 36.9\cdot kip$

$$V_{b.3b.2}^{T} = (50.6 \quad 58.8 \quad 67.1 \quad 75.3 \quad 83.5 \quad 91.7 \quad 99.9 \quad 108.1 \quad 116.3)\cdot kip$$

Nominal Shear Strength for $t_{r.e} = 6\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$$\phi V_{c.b.3a.2}^{T} = (38 \quad 44.1 \quad 50.3 \quad 56.4 \quad 62.6 \quad 68.8 \quad 74.9 \quad 81.1 \quad 87.2)\cdot kip$$

Roof Diaph. Shear Strength for $t_{r.e} = 6\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100\cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 5.9 & 6.7 & 7.5 & 8.3 & 9.1 & 9.9 & 10.7 & 11.5 & 12.2 \\ 5.9 & 6.7 & 7.5 & 8.3 & 9.1 & 9.9 & 10.7 & 11.5 & 12.2 \\ 5.9 & 6.7 & 7.5 & 8.3 & 9.1 & 9.9 & 10.7 & 11.5 & 12.2 \end{pmatrix}\cdot\%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 100\cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 3.4 & 4 & 4.6 & 5.2 & 5.8 & 6.4 & 7 & 7.6 & 8.2 \\ 3.7 & 4.4 & 5.1 & 5.7 & 6.4 & 7.1 & 7.7 & 8.4 & 9 \\ 4 & 4.7 & 5.4 & 6.1 & 6.8 & 7.5 & 8.2 & 8.9 & 9.6 \end{pmatrix}\cdot\%$$

$chk_{b.3d} = "OK"$

Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \;\; 10 \;\; 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

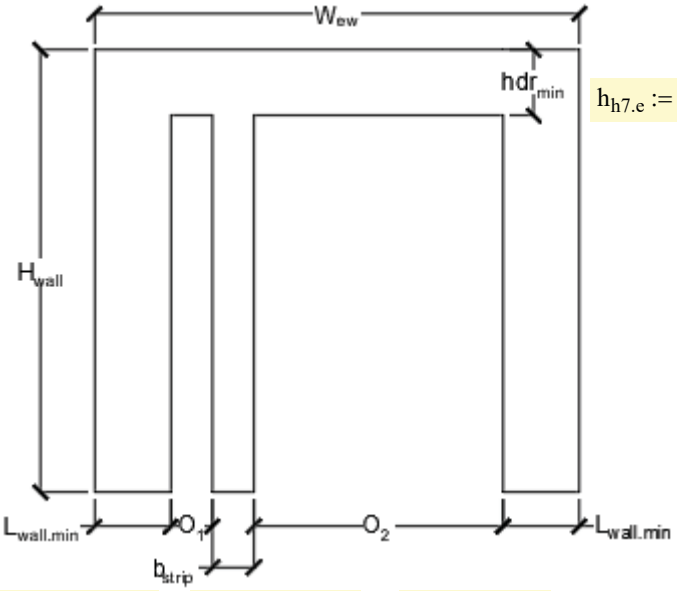
$Unity_{b.3g}^{T} = (5.8 \;\; 5.6 \;\; 5.5 \;\; 5.4 \;\; 5.4 \;\; 5.3 \;\; 5.3 \;\; 5.2 \;\; 5.2) \cdot \%$ $chk_{b.3j} = "OK"$

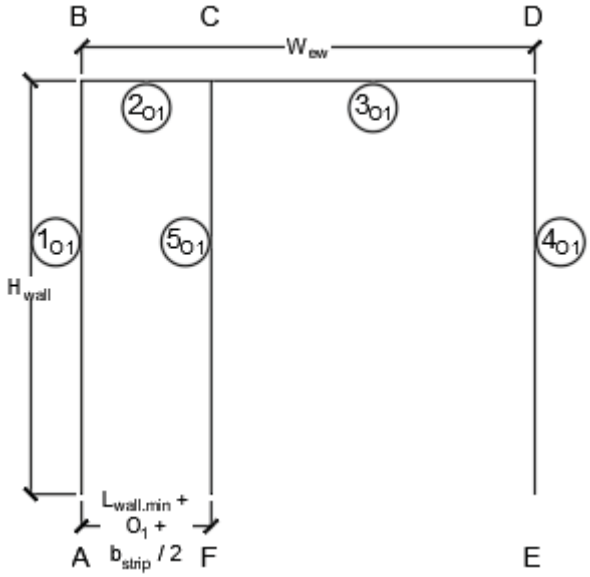
Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \;\; 10 \;\; 10.7)\,ft$ & $w_{llr} = 100 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 3.4 & 2.9 & 2.6 & 2.3 & 2.1 & 1.9 & 1.7 & 1.6 & 1.5 \\ 3.8 & 3.2 & 2.8 & 2.5 & 2.3 & 2.1 & 1.9 & 1.8 & 1.6 \\ 4 & 3.4 & 3 & 2.7 & 2.4 & 2.2 & 2 & 1.9 & 1.7 \end{pmatrix} \cdot \%$$

$chk_{b.3j} = "OK"$

END WALL DESIGN





$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} = "Modulus of Elas. Conc."$

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right)$$ $Veq_{ewho}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5) \cdot kip$ Height $H_{wall_0} = 9\,ft$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right)$$ $Veq_{ewh1}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5) \cdot kip$ Height $H_{wall_1} = 10\,ft$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right)$$ $Veq_{ewh2}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5) \cdot kip$ Height $H_{wall_2} = 10.7\,ft$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.co

Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \frac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \frac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

Joint C

$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix}$ $K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \frac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \frac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}^T = (3.7 \;\; 4.2 \;\; 4.7 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.7 \;\; 7.1 \;\; 7.6) \cdot kip \cdot ft$

$V_{L.ewh0}^T = (0.7 \;\; 0.8 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.5) \cdot kip$

$M_{hdrh0}^T = (2.7 \;\; 3.1 \;\; 3.4 \;\; 3.8 \;\; 4.2 \;\; 4.5 \;\; 4.9 \;\; 5.2 \;\; 5.6) \cdot kip \cdot ft$

$V_{hdrh0}^T = (0.9 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.6 \;\; 1.7 \;\; 1.8 \;\; 1.9) \cdot kip$

$M_{b.striph0}^T = (4.1 \;\; 4.6 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.8 \;\; 7.3 \;\; 7.9 \;\; 8.4) \cdot kip \cdot ft$

$V_{b.striph0}^T = (0.8 \;\; 0.9 \;\; 1.1 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.2 \;\; 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10\,ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}^T = (4.3 \quad 4.8 \quad 5.4 \quad 5.9 \quad 6.5 \quad 7.1 \quad 7.6 \quad 8.2 \quad 8.8) \cdot kip \cdot ft$

$M_{hdrh1}^T = (3.4 \quad 3.8 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6.1 \quad 6.5 \quad 7) \cdot kip \cdot ft$

$M_{b.striph1}^T = (4.6 \quad 5.2 \quad 5.8 \quad 6.4 \quad 7 \quad 7.6 \quad 8.2 \quad 8.8 \quad 9.4) \cdot kip \cdot ft$

$V_{L.ewh1}^T = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$V_{hdrh1}^T = (1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.5 \quad 2.7 \quad 2.9 \quad 3.1) \cdot kip$

$V_{b.striph1}^T = (0.9 \quad 1 \quad 1.1 \quad 0.8 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.7 \quad 1.8) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119 \cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

$M_{L.ewh2}^T = (4.6 \;\; 5.2 \;\; 5.8 \;\; 6.4 \;\; 7 \;\; 7.6 \;\; 8.2 \;\; 8.8 \;\; 9.4)\cdot kip\cdot ft$ $V_{L.ewh2}^T = (0.8 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.5 \;\; 1.6)\cdot kip$

$M_{hdrh2}^T = (3.7 \;\; 4.2 \;\; 4.7 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.6 \;\; 7.1 \;\; 7.6)\cdot kip\cdot ft$ $V_{hdrh2}^T = (1.6 \;\; 1.9 \;\; 2.1 \;\; 2.3 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.2 \;\; 3.4)\cdot kip$

$M_{b.striph2}^T = (4.9 \;\; 5.5 \;\; 6.2 \;\; 6.8 \;\; 7.5 \;\; 8.1 \;\; 8.7 \;\; 9.4 \;\; 10)\cdot kip\cdot ft$ $V_{b.striph2}^T = (0.9 \;\; 1 \;\; 1.1 \;\; 0.8 \;\; 1.3 \;\; 1.4 \;\; 1.5 \;\; 1.7 \;\; 1.8)\cdot kip$

Reinforcement

$B_{3.1} = \text{"\#6"}$ $B_{3.1.ref} = \text{"Wall Vert. Perim. Reinf."}$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = \text{"\# Wall Vert. Perim. Bars"}$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.3} = \text{"\#6"}$ $B_{3.3.ref} = \text{"EW Bot Perim. Reinf."}$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = \text{"\# EW Bot Perim. Bars"}$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.4} = \text{"\#4"}$ $B_{3.4.ref} = \text{"SW Bot Perim. Reinf."}$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = \text{"\# SW Bot Perim. Bars"}$

$D_{3.4} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17\cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5\cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5\cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{ewc} = 1.7\cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{h.e} = 1.6\cdot in$

$a_{strip} := \frac{A_{3.5}\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{strip} = 2.4\cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot L_{ew.mn}$ $A_{mn.e} = 0.079\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.h.e} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.strip} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w\cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w\cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w\cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

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$chk_{c.1} := if\left(A_{mn.e} \le \min\left(A_{3.1}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.1} = "OK"$

$chk_{c.2} := if\left(A_{mn.e} \le \min\left(A_{3.2}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.2} = "OK"$

$chk_{c.3} := if\left(A_{mn.strip} \le A_{3.5}, "OK", "NG"\right)$ $chk_{c.3} = "OK"$

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = "Tensile Strain, Extreme Layer"$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{ewc} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{h.e} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{strip} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi Mn_{ewc} := \phi_{ewc} \cdot \min\left(A_{3.1}, A_{3.5}\right) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min\left(A_{3.2}, A_{3.5}\right) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref} = "Flexural Strength, +ve, Sect (22.3)"$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"$

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$Vc_{ewc.1} := 8 \cdot \lambda \cdot \lambda_{s.ewc} \cdot \rho_{ewc}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975 \cdot kip$ $Vc_{ewc.2} = 19.387 \cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V \cdot \min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2 \cdot \left(t_w \cdot d_{ewc}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{ewc} = 3.7 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8 \cdot \lambda \cdot \lambda_{s.h.e} \cdot \rho_{h.e}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663 \cdot kip$ $Vc_{h.e.2} = 17.676 \cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V \cdot \min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.e} = 3.5 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8 \cdot \lambda \cdot \lambda_{s.strip}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.2} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347 \cdot kip$ $Vc_{strip.2} = 26.799 \cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V \cdot \min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2 \cdot \left(t_w \cdot d_{h7.e}\right) \cdot \sqrt{f'_c \cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3 \cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.19 & 0.2 & 0.21 & 0.12 & 0.13 & 0.14 \\ 0.22 & 0.23 & 0.23 & 0.13 & 0.15 & 0.16 \\ 0.24 & 0.26 & 0.26 & 0.15 & 0.17 & 0.18 \\ 0.27 & 0.29 & 0.29 & 0.16 & 0.19 & 0.2 \\ 0.29 & 0.31 & 0.32 & 0.18 & 0.2 & 0.22 \\ 0.32 & 0.34 & 0.34 & 0.19 & 0.22 & 0.24 \\ 0.34 & 0.37 & 0.37 & 0.21 & 0.24 & 0.26 \\ 0.37 & 0.4 & 0.4 & 0.22 & 0.26 & 0.28 \\ 0.39 & 0.42 & 0.43 & 0.24 & 0.27 & 0.29 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

$$Uny_{strip} = \begin{pmatrix} 0.1 & 0.1 & 0.1 & 0.07 & 0.07 & 0.08 \\ 0.11 & 0.12 & 0.12 & 0.07 & 0.08 & 0.09 \\ 0.13 & 0.13 & 0.13 & 0.08 & 0.09 & 0.1 \\ 0.11 & 0.1 & 0.09 & 0.09 & 0.1 & 0.11 \\ 0.12 & 0.16 & 0.16 & 0.1 & 0.11 & 0.12 \\ 0.13 & 0.17 & 0.17 & 0.11 & 0.12 & 0.13 \\ 0.14 & 0.18 & 0.19 & 0.12 & 0.13 & 0.14 \\ 0.15 & 0.2 & 0.2 & 0.13 & 0.14 & 0.15 \\ 0.16 & 0.21 & 0.21 & 0.14 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$Uny_{hdr} = \begin{pmatrix} 0.27 & 0.43 & 0.47 & 0.1 & 0.13 & 0.14 \\ 0.31 & 0.49 & 0.53 & 0.12 & 0.14 & 0.16 \\ 0.34 & 0.55 & 0.59 & 0.13 & 0.16 & 0.18 \\ 0.38 & 0.6 & 0.66 & 0.14 & 0.18 & 0.19 \\ 0.41 & 0.66 & 0.72 & 0.16 & 0.2 & 0.21 \\ 0.45 & 0.72 & 0.78 & 0.17 & 0.21 & 0.23 \\ 0.48 & 0.78 & 0.84 & 0.18 & 0.23 & 0.25 \\ 0.52 & 0.83 & 0.91 & 0.2 & 0.25 & 0.27 \\ 0.56 & 0.89 & 0.97 & 0.21 & 0.26 & 0.29 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

$Vuh1^T = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

$Vuh2^T = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Wall Plates: $No_{plates} := 1 \quad V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$

$V_{u.Floor.PLh0}{}^T = (3.4 \;\; 3.8 \;\; 4.3 \;\; 4.7 \;\; 5.2 \;\; 5.6 \;\; 6.1 \;\; 6.5 \;\; 7)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$

$V_{u.Floor.PLh1}{}^T = (3.8 \;\; 4.3 \;\; 4.8 \;\; 5.3 \;\; 5.8 \;\; 6.3 \;\; 6.8 \;\; 7.3 \;\; 7.8)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$

$V_{u.Floor.PLh2}{}^T = (4 \;\; 4.5 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.7 \;\; 7.2 \;\; 7.7 \;\; 8.3)\cdot kip$

$A_{cv.b.3b.1}{}^T = (888 \;\; 1032 \;\; 1176 \;\; 1320 \;\; 1464 \;\; 1608 \;\; 1752 \;\; 1896 \;\; 2040)\cdot in^2$

$\rho_{b.2}{}^T = (0.00077 \;\; 0.00066 \;\; 0.00058 \;\; 0.00052 \;\; 0.00046 \;\; 0.00042 \;\; 0.00039 \;\; 0.00036 \;\; 0.00033)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \quad \Longrightarrow \quad 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force $v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.1.2h0}{}^T = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.7 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.2.2h1}{}^T = (2.4 \;\; 2.8 \;\; 3.1 \;\; 3.4 \;\; 3.7 \;\; 4.1 \;\; 4.4 \;\; 4.7 \;\; 5)\cdot\dfrac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$v_{u.5d.3.2h2} := \frac{V_{u.Wall.PLh2}}{\left\lceil \left(L_{ew.mn}\right) - 3.5in \right\rceil}$$

$$v_{u.5d.3.2h2}^{T} = (2.6 \quad 2.9 \quad 3.3 \quad 3.6 \quad 4 \quad 4.3 \quad 4.7 \quad 5 \quad 5.4)\cdot\frac{kip}{ft}$$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.2.2h1}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.3.2h2}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}\right)^{T} \qquad Hwall := \left(H_{wall_0} \quad H_{wall_1} \quad H_{wall_2}\right)$$

$$Unity_{5d1.2.3}^{T} = \begin{pmatrix} 34.08 & 38.61 & 43.14 & 47.66 & 52.19 & 56.71 & 61.24 & 65.76 & 70.29 \\ 37.87 & 42.9 & 47.93 & 52.96 & 57.99 & 63.01 & 68.04 & 73.07 & 78.1 \\ 40.4 & 45.76 & 51.12 & 56.49 & 61.85 & 67.21 & 72.58 & 77.94 & 83.31 \end{pmatrix}\cdot\% \qquad Hwall^{T} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$$

$$L^{T} = (12.00 \quad 14.00 \quad 16.00 \quad 18.00 \quad 20.00 \quad 22.00 \quad 24.00 \quad 26.00 \quad 28.00)\,ft$$

$chk_{5d.1} := if\left(1.0 \geq \max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4\cdot in$ Wall Thickness $w_{plate} = 16\cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \end{pmatrix} \quad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix}\cdot in \quad b_{eff.5d.4} := b_{eff.5d_0} \quad b_{eff.5d.5} := b_{eff.5d_1} \quad b_{eff.5d.6} := b_{eff.5d_2}$$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$	$b_{5d.4} = 8.3\,ft$	For Wall Panel Height of
$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.4} := H_{wall_0} - t_{floor}$	$l_{c.5d.4} = 8.3\,ft$	$H_{wall_0} = 9\,ft$
$a_{5d5} := W_{ew}$	$a_{5d5} = 11.7\,ft$	$b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$	$b_{5d.5} = 9.3\,ft$	For Wall Panel Height of
$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.5} := H_{wall_1} - t_{floor}$	$l_{c.5d.5} = 9.3\,ft$	$H_{wall_1} = 10\,ft$
$a_{5d6} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$	$b_{5d.6} = 10\,ft$	For Wall Panel Height of
$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$	$a\_b_{5d.6} = 1.2$	$l_{c.5d.6} := H_{wall_2} - t_{floor}$	$l_{c.5d.6} = 10\,ft$	$H_{wall_2} = 10.7ft$

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \quad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft \quad P_{5.5} := \max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right) \quad P_{5.5} = 53.3\cdot psf$$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4}\cdot P_{5.5}\cdot\left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 15.08\cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6}\cdot 1\,ft$ $S_{5d.4} = 32\cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4}\cdot S_{5d.4}$ $Mu_{5d.4} = 3.4\cdot kip\cdot in$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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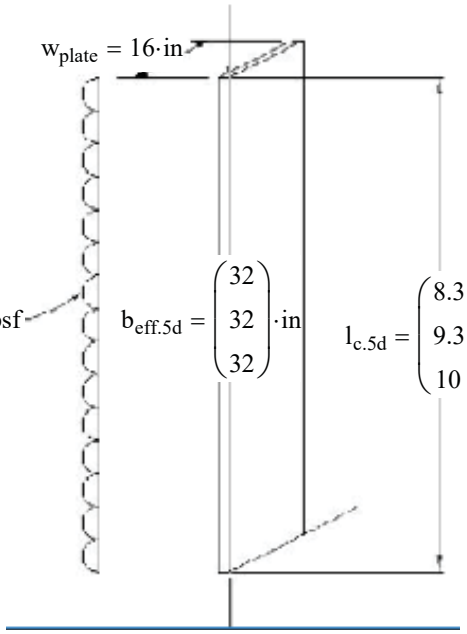
Assume only (1) end wall for resistance to lateral load $P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}^{T} = (6.8 \;\; 7.7 \;\; 8.6 \;\; 9.5 \;\; 10.4 \;\; 11.3 \;\; 12.2)$ $P_{5d.4max} := \max(2 \cdot V_{u.Wall.PLh0})$ $P_{5d.4max} = 14 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}^{T} = (3.4 \;\; 3.8 \;\; 4.3 \;\; 4.7 \;\; 5.2 \;\; 5.6 \;\; 6.1 \;\; 6.5 \;\; 7) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \dfrac{\beta_{5d.5} \cdot P_{5.5} \cdot (l_{c.5d.5})^2}{(t_w)^2} = 18.92 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \dfrac{\beta_{5d.6} \cdot P_{5.5} \cdot (l_{c.5d.6})^2}{(t_w)^2} = 17.33 \cdot ksf$



$S_{5d.5} := \dfrac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15.5 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \dfrac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$

$P_{5d.5}^{T} = (7.5 \;\; 8.5 \;\; 9.5 \;\; 10.5 \;\; 11.5 \;\; 12.5 \;\; 13.5 \;\; 14.5 \;\; 15.5) \cdot kip$

$V_{u.Floor.PLh1}^{T} = (3.8 \;\; 4.3 \;\; 4.8 \;\; 5.3 \;\; 5.8 \;\; 6.3 \;\; 6.8 \;\; 7.3 \;\; 7.8) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^{T} = (8 \;\; 9.1 \;\; 10.2 \;\; 11.2 \;\; 12.3 \;\; 13.3 \;\; 14.4 \;\; 15.5 \;\; 16.5) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.5 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^{T} = (4 \;\; 4.5 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.7 \;\; 7.2 \;\; 7.7 \;\; 8.3) \cdot kip$

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Occupancy:	S2
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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} = 232.6 \cdot kip \cdot in$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\dfrac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

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$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^{T} = (6.6 \quad 7.5 \quad 8.3 \quad 9.2 \quad 10.1 \quad 11 \quad 11.8 \quad 12.7 \quad 13.6)\cdot\%$

$chk_{5d.3} := if(1.0 \geq 2\,max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} = 289.5\cdot kip\cdot in$ $Mu_{5d.5} = 4.2\cdot kip\cdot in$

$chk_{5d.4} := if\left(P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

$\phi P_{5d.5} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.5}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.5}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9\cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^{T} = (8.8 \quad 9.9 \quad 11.1 \quad 12.2 \quad 13.4 \quad 14.6 \quad 15.7 \quad 16.9 \quad 18)\cdot\%$

$chk_{5d.5} := if(1.0 \geq 2\,max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

$P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} = 330.8\cdot kip\cdot in$ $Mu_{5d.6} = 3.9\cdot kip\cdot in$

$chk_{5d.6} := if\left(P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.6}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.6}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7\cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^{T} = (10.9 \quad 12.3 \quad 13.8 \quad 15.2 \quad 16.7 \quad 18.1 \quad 19.6 \quad 21 \quad 22.4)\cdot\%$

$chk_{5d.7} := if(1.0 \geq 2\,max(Unity_{5d.4}), "OK", "NG")$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w\cdot\rho_{min.5e.1}$ $Amin_{5e.1} = 0.096\cdot\frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^{T} = (1.8 \quad 1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_0} = 9\,ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^{T} = (2.6 \quad 2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3)\cdot kip$

$V_{L.ewh0}^{T} = (0.7 \quad 0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5)\cdot kip$

$Unity_{5e.1.h0}^{T} = (27.4 \quad 31.6 \quad 35.8 \quad 40.2 \quad 44.8 \quad 49.5 \quad 54.4 \quad 59.4 \quad 64.7)\cdot\%$

$chk_{5e.1} := if(1.0 \geq max(Unity_{5e.1.h0}), "OK", "NG")$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^{T} = (1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5 \quad 1.5)$

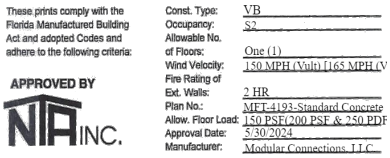
$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_1} = 10\,ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^{T} = (2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.2 \quad 2.2)\cdot kip$

$V_{L.ewh1}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6)\cdot kip$

$Unity_{5e.1.h1}^{T} = (29.8 \quad 34.4 \quad 39.1 \quad 44 \quad 49.1 \quad 54.4 \quad 59.9 \quad 65.6 \quad 71.6)\cdot\%$



$chk_{5e.2} := if\left(1.0 \geq max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^{T} = (1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5 \quad 1.5 \quad 1.5)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_2} = 10.7\,ft \qquad Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^{T} = (2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.3 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh2}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^{T} = (30.4 \quad 35.1 \quad 40 \quad 45.1 \quad 50.4 \quad 55.9 \quad 61.7 \quad 67.7 \quad 74) \cdot \%$

$chk_{5e.3} := if\left(1.0 \geq max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^{T} = (6.8 \quad 7.7 \quad 8.6 \quad 9.5 \quad 10.4 \quad 11.3 \quad 12.2 \quad 13.1 \quad 14) \cdot kip$

$Mu_{5d.4} = 3.35 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \qquad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.4} = 15.7 \qquad d_{5e.4} := \frac{t_w}{2} \qquad d_{5e.4} = 2 \cdot in \qquad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'^{T}_{5e.4} = (0.5 \quad 0.44 \quad 0.39 \quad 0.35 \quad 0.32 \quad 0.3 \quad 0.28 \quad 0.26 \quad 0.24) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \qquad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]\right]$$

$P_{d.5e.4}^{T} = (237.9 \quad 247 \quad 254.2 \quad 260.1 \quad 264.9 \quad 269 \quad 272.4 \quad 275.4 \quad 278) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \qquad Unity_{5e.4}^{T} = (2.8 \quad 3.1 \quad 3.4 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5 \quad 4.7 \quad 5) \cdot \%$

$chk_{5e.4} := if\left(1.0 \geq max\left(Unity_{5e.4}\right), "OK", "NG"\right)$ $chk_{5e.4} = "OK"$

$P_{5d.5}^{T} = (7.5 \quad 8.5 \quad 9.5 \quad 10.5 \quad 11.5 \quad 12.5 \quad 13.5 \quad 14.5 \quad 15.5) \cdot kip$

$Mu_{5d.5} = 4.2 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \qquad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.5} = 15.7 \qquad d_{5e.5} := \frac{t_w}{2} \qquad d_{5e.5} = 2 \cdot in \qquad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'^{T}_{5e.5} = (0.56 \quad 0.49 \quad 0.44 \quad 0.4 \quad 0.37 \quad 0.34 \quad 0.31 \quad 0.29 \quad 0.27) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}}\right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \qquad x_{5e.5} = 159.1 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$P_{d.5e.5} := x_{5e.5}\cdot\left[\left[-\rho_{5e.5}\cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2\cdot\frac{e'_{5e.5}}{d_{5e.5}}\cdot\rho_{5e.5}\cdot u_{5e.5}}\right]\right]$$

$P_{d.5e.5}^{T} = (227.8 \quad 238.1 \quad 246.3 \quad 252.9 \quad 258.3 \quad 262.9 \quad 266.8 \quad 270.2 \quad 273.1)\cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}}$ $\quad Unity_{5e.5}^{T} = (3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5 \quad 4.8 \quad 5.1 \quad 5.4 \quad 5.7)\cdot\%$

$chk_{5e.5} := if\left(1.0 \geq \max\left(Unity_{5e.5}\right), "OK", "NG"\right)$ $\quad chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (8 \quad 9.1 \quad 10.2 \quad 11.2 \quad 12.3 \quad 13.3 \quad 14.4 \quad 15.5 \quad 16.5)\cdot kip$

$Mu_{5d.6} = 3.85\cdot kip\cdot in$ $\quad f'_c = 4500\cdot psi$ $\quad \phi_c = 0.65$ $\quad \rho_{5e.6} := \frac{A_{s.4a}\cdot ft}{t_w\cdot b_{eff.5d.6}}$ $\quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85\cdot f'_c}$ $\quad u_{5e.6} = 15.7$ $\quad d_{5e.6} := \frac{t_w}{2}$ $\quad d_{5e.6} = 2\cdot in$ $\quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

$e'^{T}_{5e.6} = (0.48 \quad 0.42 \quad 0.38 \quad 0.34 \quad 0.31 \quad 0.29 \quad 0.27 \quad 0.25 \quad 0.23)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P_{d.5e.6}^{T} = (240.3 \quad 249.1 \quad 256.1 \quad 261.8 \quad 266.4 \quad 270.4 \quad 273.7 \quad 276.6 \quad 279.2)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity_{5e.6}^{T} = (3.3 \quad 3.6 \quad 4 \quad 4.3 \quad 4.6 \quad 4.9 \quad 5.3 \quad 5.6 \quad 5.9)\cdot\%$

$chk_{5e.6} := if\left(1.0 \geq \max\left(Unity_{5e.6}\right), "OK", "NG"\right)$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V_{u.Floor.PLh0}^{T} = (3.4 \quad 3.8 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6.1 \quad 6.5 \quad 7)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix} 794.3 \\ 833.6 \\ 859.8 \end{pmatrix}\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix} 714.8 \\ 750.2 \\ 773.8 \end{pmatrix}\cdot plf$

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx}^{T} = (794.3 \quad 833.6 \quad 859.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w_{u.5f.1.DL.mx}^{T} = (714.8 \quad 750.2 \quad 773.8)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix} 752.3 \\ 791.6 \\ 817.8 \end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{\left(H_{wall} - t_{floor}\right)}{3}$

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$w_{5f.1} = \begin{pmatrix} 2.8 \\ 3.1 \\ 3.3 \end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix} w_{DL.5f.1.mx} \\ -w_{DL.5f.1.mx} \end{pmatrix}}{No_{plates}} = \begin{bmatrix} \begin{pmatrix} 794.25 \\ 833.58 \\ 859.81 \end{pmatrix} \\ \begin{pmatrix} -794.25 \\ -833.58 \\ -859.81 \end{pmatrix} \end{bmatrix} \cdot plf$

$w_{up} := \frac{\left|P_{wind_3}\right| \cdot W_{ew}}{2}$ $w_{up} = 311.9 \cdot plf$ $w_{u.up} := w_{up}$ $w_{u.up} = 311.9 \cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h0} = (6.2 \quad 6.6 \quad 7.1 \quad 7.5 \quad 8 \quad 8.4 \quad 8.9 \quad 9.3 \quad 9.8) \cdot kip$

$P_{5f.2.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h1} = (6.4 \quad 6.9 \quad 7.3 \quad 7.8 \quad 8.2 \quad 8.7 \quad 9.1 \quad 9.6 \quad 10) \cdot kip$

$P_{5f.2.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h2} = (6.6 \quad 7 \quad 7.5 \quad 7.9 \quad 8.4 \quad 8.8 \quad 9.3 \quad 9.7 \quad 10.2) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/ 200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Tension Side

$P_{5f.3.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h0} = (-0.6 \quad -1 \quad -1.5 \quad -1.9 \quad -2.4 \quad -2.8 \quad -3.3 \quad -3.7 \quad -4.2) \cdot kip$

$P_{5f.3.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h1} = (-0.3 \quad -0.8 \quad -1.2 \quad -1.7 \quad -2.1 \quad -2.6 \quad -3 \quad -3.5 \quad -3.9) \cdot kip$

$P_{5f.3.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h2} = (-0.2 \quad -0.6 \quad -1.1 \quad -1.5 \quad -2 \quad -2.4 \quad -2.9 \quad -3.3 \quad -3.8) \cdot kip$

$W_{u.5f.1} := \left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| = 53.3 \cdot psf$ $Mu_{5f.1} := \frac{W_{u.5f.1} \cdot (H_E - 8in)^2}{8} = \begin{pmatrix} 6.4 \\ 8 \\ 9.1 \end{pmatrix} \cdot \frac{kip \cdot in}{ft}$

$d_{5.4} := 0.5 \cdot t_w$ $d_{5.4} = 2 \cdot in$ $x_{5f.2} := 0.00349$ Initial Guess for reinforcement ratio

$f(x_{5f.2}) := \frac{\frac{\max(Mu_{5f.1})}{\left(\frac{kip \cdot ft}{ft}\right)}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d_{5.4}}{in}\right)^2 \cdot (1 - 8.85 \cdot x_{5f.2})} - x_{5f.2}$ $\rho_{5f.2} := \frac{root(f(x_{5f.2}), x_{5f.2})}{ft}$ $\rho_{5f.2} = 0.00361 \cdot \frac{1}{ft}$

Reinforcement Ratio Required for out of plane Wind

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$A_{s.req1.5f.2.} := \rho_{5f.2} \cdot 12in \cdot d_{5.4}$ $A_{s.req1.5f.2.} = 0.087 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for out of plane Wind

$$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9 \cdot f_y \cdot \frac{W_{ew}}{2}}\right)$$

$A_{s.req2.5f.2} = 0.01322 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for Tension Force from Uplift

$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2}$ $A_{req.5f.2} = 0.1 \cdot \frac{in^2}{ft}$ Total Area of Reinforcement Required

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ Area of Steel Provided

$chk_{5.8} := if\left(A_{req.5f.2} \le A_{s.4a}, "OK", "NG"\right)$ $chk_{5.8} = "OK"$

Check P-Δ Effects

$E_{c.spec} = 2991 \cdot ksi$ $I_{g.5g.1} := \frac{1}{12} \cdot 12in \cdot \left(t_w\right)^3$ $I_{g.5g.1} = 64 \cdot in^4$

$P_{cr.5g.1} := \frac{\pi^2 \cdot E_{c.spec} \cdot I_{g.5g.1}}{H_E^2}$ $P_{cr.5g.1} = \begin{pmatrix} 141 \\ 115.8 \\ 102.5 \end{pmatrix} \cdot kip$ $Ma_{g.2} := \frac{\left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| \cdot \left(H_{wall}\right)^2}{8} \cdot 12in$ $Ma_{g.2} = \begin{pmatrix} 539.3 \\ 665.9 \\ 757.6 \end{pmatrix} \cdot lb \cdot ft$

$Mb_{5g.2} := Ma_{g.2}$ $Mb_{5g.2} = \begin{pmatrix} 0.5 \\ 0.7 \\ 0.8 \end{pmatrix} \cdot kip \cdot ft$ $C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}}$ $C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}} \qquad \delta_{ns.5.6} = \begin{pmatrix} 1.2 \\ 1.2 \\ 1.2 \end{pmatrix}$$

$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix}$ $Ma_{g.3} = \begin{pmatrix} 630.3 \\ 807.9 \\ 945.2 \end{pmatrix} \cdot lb \cdot ft$ $f_r = 427.6 \cdot psi$ $M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}}$ $M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking Occurs", "Cracking Doesn't Occur"\right)$ $chk_{5.9} = "Cracking Doesn't Occur"$

$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}}$ $\Delta_{5g.1} = \begin{pmatrix} 0.18 \\ 0.24 \\ 0.28 \end{pmatrix} \cdot in$ $Mu_{5.5} := Mu_{5f.1} \cdot ft$ $Mu_{5.5} = \begin{pmatrix} 6.4 \\ 8 \\ 9.1 \end{pmatrix} \cdot kip \cdot in$

$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$

$Mu_{5.6} = (10.2 \quad 10.4 \quad 10.5 \quad 10.7 \quad 10.8 \quad 11 \quad 11.2 \quad 11.3 \quad 11.5) \cdot kip \cdot in$

$P_{5.8} := Embed_{5f.1}$

$P_{5.8} = (3.4 \quad 3.8 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6.1 \quad 6.5 \quad 7) \cdot kip$

$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}}$ $e`_{5.6} = (3 \quad 2.7 \quad 2.5 \quad 2.3 \quad 2.1 \quad 2 \quad 1.8 \quad 1.7 \quad 1.6) \cdot in$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$u_{5.} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f'_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

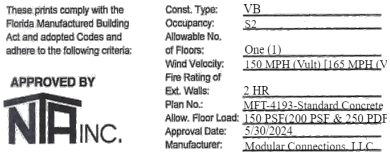
$$\rho_{5.8} := \frac{A_{s.4a}}{\max(w_{5f.1}) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max(e`_{5.6})}{d_{5.4}} + \sqrt{\left(1 - \frac{\max(e`_{5.6})}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max(e`_{5.6}) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 38.4 \\ 43.1 \\ 46.1 \end{pmatrix} \cdot kip$$

$chk_{5.10} := \text{if}(\max(P_{5.8}) \le \min(\phi P_{5.6}), \text{"OK"}, \text{"NG"})$ $\qquad chk_{5.10} = \text{"OK"}$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"



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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 41.1 \\ 40.4 \\ 39.8 \\ 39.4 \\ 39.1 \\ 38.8 \\ 38.6 \\ 37.8 \\ 37.7 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 12 ' "} \\ \text{"Min Flr Uniform Weight, for L = 14 ' "} \\ \text{"Min Flr Uniform Weight, for L = 16 ' "} \\ \text{"Min Flr Uniform Weight, for L = 18 ' "} \\ \text{"Min Flr Uniform Weight, for L = 20 ' "} \\ \text{"Min Flr Uniform Weight, for L = 22 ' "} \\ \text{"Min Flr Uniform Weight, for L = 24 ' "} \\ \text{"Min Flr Uniform Weight, for L = 26 ' "} \\ \text{"Min Flr Uniform Weight, for L = 28 ' "} \end{pmatrix}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$wg_{f.mx} = \begin{pmatrix} 45.1 \\ 44.4 \\ 43.8 \\ 43.4 \\ 43.1 \\ 42.8 \\ 42.6 \\ 41.8 \\ 41.7 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf \quad \text{Wall Weight} \qquad w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf \quad \text{Wall Misc. Weight}$$

$DL_{r.mx} = 73.4 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 66.4 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -53.48 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 46 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.183$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

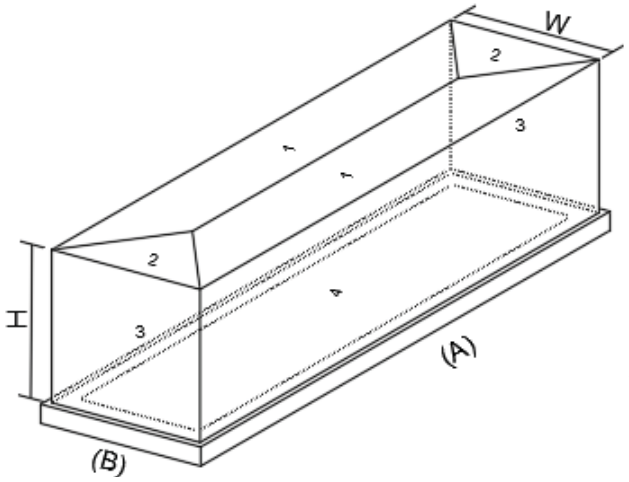
$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.147$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\ ft^2$

$A_f^T = (123.74 \quad 145.65 \quad 167.57 \quad 189.49 \quad 211.4 \quad 233.32 \quad 255.24 \quad 277.15 \quad 299.07)\ ft^2$

$L_{wall}^T = (45.92 \quad 49.92 \quad 53.92 \quad 57.92 \quad 61.92 \quad 65.92 \quad 69.92 \quad 73.92 \quad 77.92)\ ft$



$$DL_{A.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) \ldots + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 52.01\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 46.06\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\left[\begin{matrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor} \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{matrix}\right]}{\begin{matrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right] \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{matrix}}\right] \qquad \begin{matrix} z_{mx} = 67.88\cdot in \\ z_{mx} = 5.66\cdot ft\end{matrix}$$

$ftg_{exp} := 36in$ Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.81\,ft$ Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\,ft$ Moment Arm for Floor Level

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$DL_{A.f0.mx}^{T} = (1167.4 \;\; 1163.3 \;\; 1160.3 \;\; 1157.9 \;\; 1156.1 \;\; 1154.6 \;\; 1153.3 \;\; 1149 \;\; 1148.3)\cdot plf$

$DL_{A.f0.mn}^{T} = (1343.5 \;\; 1284.3 \;\; 1240.6 \;\; 1207.1 \;\; 1180.4 \;\; 1158.8 \;\; 1140.9 \;\; 1122.6 \;\; 1109.9)\cdot plf$

$DL_{B.f0.mx}^{T} = (873.9 \;\; 869.8 \;\; 866.8 \;\; 864.4 \;\; 862.6 \;\; 861.1 \;\; 859.8 \;\; 855.5 \;\; 854.8)\cdot plf$

$DL_{B.f0.mn}^{T} = (1078 \;\; 1018.8 \;\; 975.1 \;\; 941.6 \;\; 914.9 \;\; 893.3 \;\; 875.4 \;\; 857.1 \;\; 844.4)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1660 \;\; 1656 \;\; 1653 \;\; 1650 \;\; 1649 \;\; 1647 \;\; 1646 \;\; 1641 \;\; 1641)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (1343 \;\; 1284 \;\; 1241 \;\; 1207 \;\; 1180 \;\; 1159 \;\; 1141 \;\; 1123 \;\; 1110)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (873 \;\; 872 \;\; 871 \;\; 870 \;\; 869 \;\; 869 \;\; 868 \;\; 867 \;\; 867)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (612 \;\; 611 \;\; 610 \;\; 609 \;\; 608 \;\; 608 \;\; 607 \;\; 606 \;\; 606)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C} \qquad E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

$E_{O.A.E.f0.C}^{T} = (282 \;\; 281.3 \;\; 280.8 \;\; 280.4 \;\; 280.1 \;\; 279.9 \;\; 279.6 \;\; 278.9 \;\; 278.8)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-282 \;\; -281.3 \;\; -280.8 \;\; -280.4 \;\; -280.1 \;\; -279.9 \;\; -279.6 \;\; -278.9 \;\; -278.8)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (144 \;\; 122.1 \;\; 106 \;\; 93.7 \;\; 83.9 \;\; 76 \;\; 69.4 \;\; 63.8 \;\; 59.1)\cdot plf$

$E_{O.B.E.f0.T}^{T} = (-144 \;\; -122.1 \;\; -106 \;\; -93.7 \;\; -83.9 \;\; -76 \;\; -69.4 \;\; -63.8 \;\; -59.1)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (270.4 \;\; 269.7 \;\; 269.2 \;\; 268.8 \;\; 268.5 \;\; 268.2 \;\; 268 \;\; 267.3 \;\; 267.2)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-270.4 \;\; -269.7 \;\; -269.2 \;\; -268.8 \;\; -268.5 \;\; -268.2 \;\; -268 \;\; -267.3 \;\; -267.2)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (140.2 \;\; 118.9 \;\; 103.2 \;\; 91.2 \;\; 81.7 \;\; 74 \;\; 67.6 \;\; 62.1 \;\; 57.6)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-140.2 \;\; -118.9 \;\; -103.2 \;\; -91.2 \;\; -81.7 \;\; -74 \;\; -67.6 \;\; -62.1 \;\; -57.6)\cdot plf$

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Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$E_{O.A.E.f2.C}^{T} = (258.8 \;\; 258.1 \;\; 257.5 \;\; 257.1 \;\; 256.8 \;\; 256.6 \;\; 256.4 \;\; 255.6 \;\; 255.5)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-258.8 \;\; -258.1 \;\; -257.5 \;\; -257.1 \;\; -256.8 \;\; -256.6 \;\; -256.4 \;\; -255.6 \;\; -255.5)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (136.5 \;\; 115.7 \;\; 100.5 \;\; 88.8 \;\; 79.5 \;\; 72 \;\; 65.8 \;\; 60.5 \;\; 56)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-136.5 \;\; -115.7 \;\; -100.5 \;\; -88.8 \;\; -79.5 \;\; -72 \;\; -65.8 \;\; -60.5 \;\; -56)\cdot plf$

$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$

$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$

$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$

$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$

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$E_{8n.A.f0}^{T} = (829.8 \;\; 827.7 \;\; 826.2 \;\; 825 \;\; 824.1 \;\; 823.4 \;\; 822.7 \;\; 820.6 \;\; 820.2)\cdot plf$

$E_{8n.A.f0}^{T} = (-232.8 \;\; -234.2 \;\; -235.3 \;\; -236.2 \;\; -236.8 \;\; -237.4 \;\; -237.8 \;\; -237.7 \;\; -238.1)\cdot plf$

$E_{8n.B.f0}^{T} = (432.1 \;\; 409.8 \;\; 393.4 \;\; 380.8 \;\; 370.8 \;\; 362.7 \;\; 356 \;\; 349.9 \;\; 345.2)\cdot plf$

$E_{8n.B.f0}^{T} = (-121.5 \;\; -99.7 \;\; -83.7 \;\; -71.4 \;\; -61.6 \;\; -53.7 \;\; -47.2 \;\; -41.6 \;\; -36.9)\cdot plf$

$E_{8n.A.f1}^{T} = (795.6 \;\; 793.5 \;\; 792 \;\; 790.8 \;\; 789.9 \;\; 789.1 \;\; 788.5 \;\; 786.3 \;\; 786)\cdot plf$

$E_{8n.B.f1}^{T} = (420.8 \;\; 399.1 \;\; 383.1 \;\; 370.8 \;\; 361.1 \;\; 353.2 \;\; 346.7 \;\; 340.7 \;\; 336.1)\cdot plf$

$E_{8n.A.f1}^{T} = (-214.4 \;\; -213.8 \;\; -213.4 \;\; -213.1 \;\; -212.9 \;\; -212.7 \;\; -212.5 \;\; -211.9 \;\; -211.8)\cdot plf$

$E_{8n.B.f1}^{T} = (-109.8 \;\; -88.6 \;\; -72.9 \;\; -60.9 \;\; -51.4 \;\; -43.7 \;\; -37.4 \;\; -32 \;\; -27.4)\cdot plf$

$E_{8n.B.f2}^{T} = (409.5 \;\; 388.4 \;\; 372.8 \;\; 360.8 \;\; 351.3 \;\; 343.7 \;\; 337.3 \;\; 331.5 \;\; 327)\cdot plf$

$E_{8n.A.f2}^{T} = (-205.3 \;\; -204.7 \;\; -204.3 \;\; -204 \;\; -203.8 \;\; -203.6 \;\; -203.4 \;\; -202.8 \;\; -202.7)\cdot plf$

$E_{8n.A.f2}^{T} = (761.3 \;\; 759.3 \;\; 757.7 \;\; 756.6 \;\; 755.6 \;\; 754.9 \;\; 754.3 \;\; 752.1 \;\; 751.8)\cdot plf$

$E_{8n.B.f2}^{T} = (-106.9 \;\; -86.2 \;\; -71 \;\; -59.3 \;\; -50.1 \;\; -42.6 \;\; -36.4 \;\; -31.1 \;\; -26.7)\cdot plf$

$LL_{r.A.} := \frac{W_r}{2}\cdot(w_{llr})$ $\quad LL_{r.A.} = 600\cdot plf$ $\quad LL_{r.B.} := \frac{W_r}{6}\cdot(w_{llr})$ $\quad LL_{r.B.} = 200\cdot plf$

$LL_{A.f0} := \frac{W_f}{2}\cdot(w_{llf.0})$ $\quad LL_{A.f0} = 1369.79\cdot plf$ $\quad LL_{B.f0} := s_{t.rib}\cdot(w_{llf.0})$ $\quad LL_{B.f0} = 500\cdot plf$

$LL_{A.f1} := \frac{W_f}{2}\cdot(w_{llf.1})$ $\quad LL_{A.f1} = 1095.83\cdot plf$ $\quad LL_{B.f1} := s_{t.rib}\cdot(w_{llf.1})$ $\quad LL_{B.f1} = 400\cdot plf$

$LL_{A.f2} := \frac{W_f}{2}\cdot(w_{llf.2})$ $\quad LL_{A.f2} = 821.88\cdot plf$ $\quad LL_{B.f2} := s_{t.rib}\cdot(w_{llf.2})$ $\quad LL_{B.f2} = 300\cdot plf$

$SL_A := \frac{W_r}{2}\cdot(p_f)$ $\quad SL_A = 190.51\cdot plf$ $\quad SL_B := \frac{W_r}{6}\cdot(p_f)$ $\quad SL_B = 63.5\cdot plf$

$W_{up.A} := \frac{W_r}{2}\cdot P_{wind_3}$ $\quad W_{up.A} = -320.86\cdot plf$ $\quad W_{up.B} := \frac{W_r}{6}\cdot P_{wind_3}$ $\quad W_{up.B} = -106.95\cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $\quad W_{OT.A.C} = 618.5\cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $\quad W_{OT.B.C} = 600.2\cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $\quad W_{OT.A.T} = -618.5\cdot plf$ $\quad W_{OT.B.T} := -W_{OT.B.C}$ $\quad W_{OT.B.T} = -600.2\cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $\quad W_{A.mx} = 297.6\cdot plf$ $\quad W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $\quad W_{B.mx} = 493.3\cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $\quad W_{A.mn} = -939.4\cdot plf$ $\quad W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $\quad W_{B.mn} = -707.2\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \qquad Load_{f0} = \begin{pmatrix} 1167.43 \\ 1109.93 \\ 1369.79 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 829.79 \\ -238.09 \end{pmatrix} \cdot plf$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f0} := asd \cdot Load_{f0} \qquad LC_{8.f0} = \begin{pmatrix} 2537 \\ 1767 \\ 2795 \\ 1346 \\ 604 \\ 2718 \\ 1790 \\ 845 \\ 102 \\ 1748 \\ 1001 \\ 2773 \\ 2213 \\ 1247 \\ 499 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 13.41 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 14 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f0} := \max(15in, w_{8.f0.min}) \qquad Fdn_{W.f0} = 15 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 15 \cdot in$

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consider $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \qquad Load_{f1} = \begin{pmatrix} 1167.43 \\ 1109.93 \\ 1095.83 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 795.56 \\ -214.39 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f1} := asd \cdot Load_{f1} \qquad LC_{8.f1} = \begin{pmatrix} 2263 \\ 1767 \\ 2589 \\ 1346 \\ 604 \\ 2513 \\ 1585 \\ 845 \\ 102 \\ 1724 \\ 1017 \\ 2550 \\ 2020 \\ 1223 \\ 516 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 12.43 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 13 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f1} := \max(14in, w_{8.f1.min}) \qquad Fdn_{W.f1} = 14 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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consider $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \qquad Load_{f2} = \begin{pmatrix} 1167.43 \\ 1109.93 \\ 821.88 \\ 0 \\ 600 \\ 190.51 \\ 297.65 \\ -939.36 \\ 761.32 \\ -205.27 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f2} := asd \cdot Load_{f2} \qquad LC_{8.f2} = \begin{pmatrix} 1989 \\ 1767 \\ 2384 \\ 1346 \\ 604 \\ 2307 \\ 1379 \\ 845 \\ 102 \\ 1700 \\ 1024 \\ 2326 \\ 1819 \\ 1199 \\ 522 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 11.44 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 12 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f2} := \max(14in, w_{8.f2.min}) \qquad Fdn_{W.f2} = 14 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr} = 100 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 14 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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c. Slab on Grade Foundation

1.) Criteria: use slab on grade, $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.long}$ $A_{s.long.prov} = 0.39 \cdot in^2$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if(A_{s.long.prov} \geq A_{s.long}, "OK", "NG")$ $chk_{8.1} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp}$ $A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4$ $Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.temp}$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG")$ $chk_{8.2} = "OK"$

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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 66.375 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 73.4 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 100 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 46 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 35.8 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 32.3 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 53.5 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 3.03 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 3.035 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 7.283 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.027 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.027 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.312 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.312 \cdot klf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SEISMIC LOAD PARAMETERS: *For worst case seismic load in E-W direction, look at longest, widest shelter.*

Seismic Coefficient

$C_s = 0.183$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 33.45\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 31.07\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 27.08\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 22.85\cdot kip$

Total Seismic Shear at Roof

$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$ $V_{smaxW\_rf} = 8.614\cdot kip$

$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$ $V_{sminW\_rf} = 7.79\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$ $v_{EW\_smaxW} = 0.359\cdot klf$

$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$ $v_{EW\_sminW} = 0.139\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$ $v_{NS\_smaxW} = 0.369\cdot klf$

$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$ $v_{NS\_sminW} = 0.334\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

k_c — Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ — Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ — Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ — Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} — Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} — Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ — Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ — Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ — Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix}$ = ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ — Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.044 \cdot klf$ — Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.369 \cdot klf$ — Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.16 \cdot klf$ — Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.241 \cdot klf$ — Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.094 \cdot klf$ — Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.312 \cdot klf$ — Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 7$

$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1}$ $s_{REW} = 18.67\cdot in$

$s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$ $s_{EWR} = 19.33\cdot in$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, \min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, \min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$	$Vu_{o.s.REW} = 0.024\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$	$Vu_{i.s.REW} = 0.67\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$	$Vu_{o.w.REW} = 0.171\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$	$Vu_{i.w.REW} = 0.566\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.REW} := F_{7.1}\cdot\max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$	$Tu_{T.REW} = 0\cdot kip$	0.9DL + 1.0W	T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$	$Vu_{o.s.EWR} = 0.025\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$	$Vu_{i.s.EWR} = 0.694\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.EWR} := F_{7.2}\cdot\max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$	$Vu_{o.w.EWR} = 0.177\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$	$Vu_{i.w.EWR} = 0.587\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$	$Tu_{T.EWR} = 0.177\cdot kip$	0.9DL + 1.0W	T, roof uplift

▶ Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 6\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 6 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 135 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 6.86 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 6 \cdot in$ $h_{min.7.1a} = 6 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 161 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 4.51 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 5.145 \\ 3.38 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min\left(\left(0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1}\right)\right)$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.203 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.201 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.026$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.296 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.297 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.REW.1} = 0.203 \cdot in$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.REW.2} = 0.187 \cdot in$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2}$$ $l_{w\_Rqrd.REW.3} = 2.5 \cdot in$

$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right)$ $l_{w\_reqd.REW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.044\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.359\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.321\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.44\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.188\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.027\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 36\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.3} := \text{if}(s_{L.7.3} = 0in, s_{w.7.3}, \text{if}(s_{w.7.3} = 0in, s_{L.7.3}, \min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := \text{if}(s_{L.7.4} = 0in, s_{w.7.4}, \text{if}(s_{w.7.4} = 0in, s_{L.7.4}, \min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.155\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.377\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 0.658\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.095\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.155\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.377\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 0.658\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.095\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

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Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.209 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.254 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.204 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.24 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.119\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.195\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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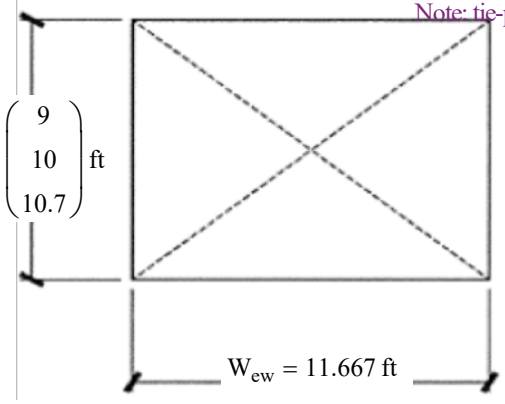
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 7$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft$$

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (13.667 \quad 15.667 \quad 17)\cdot in$ $s_{SWEW}^{T} = (13.667 \quad 15.667 \quad 17)\cdot in$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6\,ft$ Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 5.275\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{..}\cdot L_{..}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (1.208 \quad 1.039 \quad 0.912 \quad 0.812 \quad 0.732 \quad 0.667 \quad 0.612 \quad 0.566 \quad 0.526)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.007\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 1.034\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10\,ft$

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

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$M_{ot\_w.EWSW}^{T} = (33 \quad 35.9 \quad 38.9 \quad 41.9 \quad 44.9 \quad 47.9 \quad 50.9 \quad 53.9 \quad 56.9)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r + \frac{\max\left[\left(\omega_{wall}\right)\cdot\left[\max\left(H_{wall}\right)\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (51.5 \quad 55.8 \quad 60.2 \quad 64.5 \quad 68.8 \quad 73.2 \quad 77.5 \quad 81.8 \quad 86.1)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (47.6 \quad 52.9 \quad 58.2 \quad 63.4 \quad 68.7 \quad 74 \quad 79.3 \quad 84.6 \quad 89.9)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (-9.87 \quad -11.64 \quad -13.42 \quad -15.19 \quad -16.96 \quad -18.73 \quad -20.5 \quad -22.27 \quad -24.04)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (3.93 \quad 2.97 \quad 2.01 \quad 1.05 \quad 0.09 \quad -0.87 \quad -1.83 \quad -2.79 \quad -3.75)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max\left(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net}\right)$ $M_{ot.EWSW} = 3.9\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0.34\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := if\left(s_{L.7.5} = 0in, s_{w.7.5}, if\left(s_{w.7.5} = 0in, s_{L.7.5}, \min\left(s_{L.7.5}, s_{w.7.5}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := if\left(s_{L.7.6} = 0in, s_{w.7.6}, if\left(s_{w.7.6} = 0in, s_{L.7.6}, \min\left(s_{L.7.6}, s_{w.7.6}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0.056\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.334\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.334\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wall (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0.056 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.334 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.334 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.078$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.144$ < 1.0

$Result_{EWSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW}$ = "not required" < 1.2

$chk_{6.EWSW}$ = "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.079$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.239$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.14 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.14 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right) \qquad l_{w\_reqd.EWSW} = 2.5 \cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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****

e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.71 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.309 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.022 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.369 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.312 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.165 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.094 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\,ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

$M_{ot.s.FEW}^{T} = (37.3 \quad 43.4 \quad 49.5 \quad 55.6 \quad 61.7 \quad 67.8 \quad 73.9 \quad 80 \quad 86.1)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix} 0.5\cdot(L_0)^2 \\ 0.5\cdot(L_1)^2 \\ 0.5\cdot(L_2)^2 \\ 0.5\cdot(L_3)^2 \\ 0.5\cdot(L_4)^2 \\ 0.5\cdot(L_5)^2 \\ 0.5\cdot(L_6)^2 \\ 0.5\cdot(L_7)^2 \\ 0.5\cdot(L_8)^2 \end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (93.3 \quad 114.4 \quad 138.7 \quad 166.3 \quad 197.1 \quad 231.1 \quad 268.5 \quad 309 \quad 352.8)\cdot kip\cdot ft$

$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$

OT Moment, resisting

$M_{res.FEW}^{T} = (323.5 \quad 377.4 \quad 431.3 \quad 485.2 \quad 539.2 \quad 593.1 \quad 647 \quad 700.9 \quad 754.8)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-197.9 \quad -225.3 \quad -249.5 \quad -270.5 \quad -288.2 \quad -302.6 \quad -313.8 \quad -321.8 \quad -326.5)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = (-286.2 \quad -334 \quad -381.8 \quad -429.6 \quad -477.4 \quad -525.2 \quad -573 \quad -620.9 \quad -668.7)\cdot kip\cdot ft$

$M_{ot.FEW} := \max(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net})$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right]$ $\quad v_{ot.w.FEW} = 0\cdot kip$

$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right)$ $\quad v_{ot.s.FEW} = 0\cdot kip$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor

$Plate_{7.8}$ = "#2" at inside face of end wall

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

No. of Plates & maximum plate spacing $No_{pl.FEW} = 7$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \qquad s_{F\_EW} = 15.92 \cdot in$$

$$s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1} \qquad s_{EW\_F} = 17.33 \cdot in$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$$s_{7.7} := if\left(s_{L.7.7} = 0in, s_{w.7.7}, if\left(s_{w.7.7} = 0in, s_{L.7.7}, \min\left(s_{L.7.7}, s_{w.7.7}\right)\right)\right) = 3 \cdot in \qquad s_{7.7} = 3 \cdot in$$

$$F_{7.7} := 0.5 + \frac{0.5 s_{7.7} + 2 e_{PL.F\_EW}}{s_{7.7}} \qquad F_{7.7} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.7

$$s_{7.8} := if\left(s_{L.7.8} = 0in, s_{w.7.8}, if\left(s_{w.7.8} = 0in, s_{L.7.8}, \min\left(s_{L.7.8}, s_{w.7.8}\right)\right)\right) = 3 \cdot in \qquad s_{7.7} = 3 \cdot in$$

$$F_{7.8} := 0.5 + \frac{0.5 s_{7.8} + 2 e_{PL.F\_EW}}{s_{7.8}} \qquad F_{7.8} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.8

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7} \cdot \left(1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{\max(H_{wall})}{4} \cdot W_{ew}\right) \qquad Vu_{i.s.FEW} = 0.909 \cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7} \cdot 1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{i.w.FEW} = 0.607 \cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$$Tu_{po.s1.FEW} := F_{7.7} \cdot 0.3 C_s \cdot \left(t_{o.s.FEW} \cdot W_{ew}\right) \qquad Tu_{po.s1.FEW} = 0.02 \cdot kip$$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} \qquad Tu_{po.w.FEW} = 0.366 \cdot kip$$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7} \cdot \left(0.9 v_{DL.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} \qquad Vu_{o.9dl.FEW} = 1.242 \cdot kip$$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}} \qquad V_{ot.w.FEW} = 0 \cdot kip$$ 1.0W

$$V_{ot.s.FEW} := F_{7.7} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}} \qquad V_{ot.s.FEW} = 0 \cdot kip$$

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment. 1.0E

$$Vu_{o.wup.FEW} := F_{7.7} \cdot \max\left[\left(1.0 v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip\right]$$

$Vu_{o.wup.FEW} = 0 \cdot kip$ 0.9D+1.0W

$$Vu_{o.sup.FEW} := F_{7.7} \cdot \max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$$

Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.sup.FEW} = 0 \cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6 v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$$

$Vu_{o.12D.16L.FEW} = 2.617 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

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Factored loads - Floor - Endwall (Floor Wall Plates)

$Tu_{po.w.EWF} := F_{7.8} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Tu_{po.w.EWF} = 0.366 \cdot kip$ pullout, wind suction at bot of wall and bot half of two sides of wall

$Tu_{po.s2.EWF} := F_{7.8} \cdot 0.3 C_s \cdot (t_{o.s.FEW} \cdot W_{ew})$ $Tu_{po.s2.EWF} = 0.02 \cdot kip$ 0.3E pullout, seismic

$Vu_{i.s.EWF} := F_{7.8} \cdot \left(1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{\max(H_{wall})}{4} \cdot W_{ew} \right)$

$Vu_{i.s.EWF} = 0.909 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w2.EWF} := F_{7.8} \cdot 1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{i.w2.EWF} = 0.607 \cdot kip$ 1.0W in-plane V, wind roof diaphragm

$Vu_{o.9dl.EWF} := F_{7.8} \cdot (0.9 v_{DL.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{o.9dl.EWF} = 1.242 \cdot kip$ 0.9D Shear from DL only

$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - (1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 1.129 \cdot kip$ 0.9D+1.0W

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[(1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sdn.EWF} := F_{7.8} \cdot (Vu_{o.9dl.FEW} + V_{ot.s.FEW})$ $Vu_{o.sdn.EWF} = 1.45 \cdot kip$ 0.9D+1.0E

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot \left[1.2(v_{DL.FEW}) + 1.6 v_{LL.FEW} \right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 2.617 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$
$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75\cdot V_{cbg\_op.7.7} \\ 0.75\cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$$\phi P_{pl.F\_EW} = \min\left(\left(0.75\cdot N_{sa.7.7} \quad 0.75\cdot N_{cbg.7.7} \quad 0.7\cdot N_{pn.7.7}\right)\right) \qquad \phi P_{pl.F\_EW} = 8.935\cdot kip$$

Plate Summary

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041 \quad \text{or} \quad \left[\frac{Vu_{o.wup.FEW}}{0.75\cdot V_{cbg\_op.7.7}} + \frac{Vu_{i.w.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.124 \quad < 1.0$$

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.041 \quad \text{or} \quad \left[\frac{Vu_{o.sup.FEW}}{\left(0.75\cdot V_{cbg\_op.7.7}\right)} + \frac{Vu_{i.s.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.186 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.769 \quad < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8} \cdot A_{SE.7.8} \cdot f_{uta}$ $V_{sa.7.8} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719 \cdot in$ $h_{min.7.8} = 4 \cdot in$ $c_{a1\_op.7.8} = 6.5 \cdot in$ $c_{a2\_op.7.8} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90 \cdot in^2$ $A_{Vco\_op.7.8} = 190 \cdot in^2$ $e'_{V.7.8} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0 \cdot in$ $s2_{7.8a} = 3 \cdot in$ $l_{e.7.8} = 1.719 \cdot in$ $h_{ef.7.8} = 1.719 \cdot in$ $h_{a.7.8} = 4 \cdot in$ $h_{min.7.8a} = 4 \cdot in$

$c_{a1\_ip.7.8} = 13.5 \cdot in$ $c_{a2\_ip.7.8} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107 \cdot in^2$ $A_{Vco\_ip.7.8} = 820 \cdot in^2$ $e'_{V.7.8a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8} \cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75 \cdot V_{cbg\_op.7.8} \\ 0.75 \cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\left(0.75 \cdot N_{sa.7.8} \quad 0.75 \cdot N_{cbg.7.8} \quad 0.7 \cdot N_{pn.7.8}\right)\right)$ $\phi P_{pl.EW\_F} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.004 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.826 \quad < 1.0$$

$$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.085 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.598 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75 \cdot V_{cbg\_op.7.8}}\right) = 0.763 \quad < 1.0$$

$Result_{EW\_F}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F}$ = "not required" < 1.2

$chk_{6.EW\_F}$ = "OK"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Weld Between $Plate_{7.7}$ = "#1B" and $Plate_{7.8}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.1} = 0.208\cdot in$$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.2} = 0.266\cdot in$$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.3} = 0.391\cdot in$$

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^2 + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^2 + Vu_{i.s.EWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.501\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.501\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$$l_{w\_reqd.EWSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $v_{dl.FSW} = 0.924\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $v_{ll.FSW} = 0.6\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $t_{o.s.FSW} = 0.044\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $v_{i.s.FSW} = 0.359\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $v_{i.w.SW} = 0.0271\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $v_{up.w.FSW} = 0.321\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $t_{po.w.FSW} = 0.172\cdot klf$ Wind load suction on bottom of side wall

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^{T} = (93.3 \quad 114.4 \quad 138.7 \quad 166.3 \quad 197.1 \quad 231.1 \quad 268.5 \quad 309 \quad 352.8)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^{T} = (37.3 \quad 43.4 \quad 49.5 \quad 55.6 \quad 61.7 \quad 67.8 \quad 73.9 \quad 80 \quad 86.1)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^{T} = (323.5 \quad 377.4 \quad 431.3 \quad 485.2 \quad 539.2 \quad 593.1 \quad 647 \quad 700.9 \quad 754.8)\cdot kip\cdot ft$

$$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \quad v_{ot.w.FSW} = 0\cdot kip$$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

$$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right) \quad v_{ot.s.FSW} = 0\cdot kip$$

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9}$ = "#1B" at top of floor $Plate_{7.10}$ = "#2" at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 15\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.9} := if(s_{L.7.9} = 0in, s_{w.7.9}, if(s_{w.7.9} = 0in, s_{L.7.9}, min(s_{L.7.9}, s_{w.7.9}))) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}}$ $F_{7.9} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.9

$s_{7.10} := if(s_{L.7.10} = 0in, s_{w.7.10}, if(s_{w.7.10} = 0in, s_{L.7.10}, min(s_{L.7.10}, s_{w.7.10}))) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}}$ $F_{7.10} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$	$Tu_{o.w.FSW} = 0.251\cdot kip$	1.0W	T, wall suction
$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$	$Tu_{o.s.FSW} = 0.065\cdot kip$	0.3W	T, pullout seismic
$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$	$Vu_{i.s.FSW} = 0.157\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$	$Vu_{i.w.FSW} = 0.04\cdot kip$	1.0W	in-plane V roof diaphragm
$Vu_{o\_09dl.FSW} := (0.9v_{dl.FSW})\cdot s_{max.FSW}$	$Vu_{o\_09dl.FSW} = 1.039\cdot kip$	0.9D	Shear from DL only

$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.FSW} = 0\cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.FSW} = 0\cdot kip$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.FSW} \qquad +-V_{ot\_w.FSW}$$

$Vu_{o.w.FSW} = 1.68\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$$

$Vu_{o.s.FSW} = 1.212\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$$

$Vu_{o.12D.16L.FSW} = 3.017\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.251\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.065\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.SWF} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.157\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.04\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.039\cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \dfrac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0\cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \dfrac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0\cdot kip$

$$Vu_{o.w.SWF} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.SWF} \qquad +-V_{ot\_w.SWF}$$

$Vu_{o.w.SWF} = 1.68\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$$Vu_{o.s.SWF} := F_{7.10}\cdot\max\left(V_{ot\_s.SWF} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$$

$Vu_{o.s.SWF} = 1.212\cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.SWF} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$$

$Vu_{o.12D.16L.SWF} = 3.017\cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5\cdot in$ $w_{pl.7.9} = 5\cdot in$ $x_{Le.7.9} = 1\cdot in$ $x_{we.7.9} = 4\cdot in$ $t_{pl.7.9} = 0.313\cdot in$

$h_{ef.7.9} = 3.719\cdot in$ $h_{a.7.9} = 8\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0\cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5\cdot in$ $A_{SE.7.9} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9}\cdot A_{SE.7.9}\cdot f_{uta}$ $N_{sa.7.9} = 70\cdot kip$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.028$ or $\left[\frac{Vu_{o.w.FSW}}{(0.75 \cdot V_{cbg\_op.7.9})} + \frac{Vu_{i.w.FSW}}{(0.75 \cdot V_{cbg\_ip.7.9})}\right] = 0.503 < 1.0$

$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.007$ or $\left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{(0.75 \cdot V_{cbg\_ip.7.9})}\right] = 0.391 < 1.0$

$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.887 < 1.0$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.058 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.505 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.015 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.418 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.876 < 1.0$$

$Result_{SWF}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWF}$ = "not required" < 1.2

$chk_{6.SWF}$ = "OK"

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.1} = 0.498 \cdot in$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.2} = 0.359 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.498\cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.359\cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.149\cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min(w_{pl.7.9}, w_{pl.7.10})}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5\cdot in$$

$l_{w\_reqd.FSW} := \max(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6})$ $l_{w\_reqd.FSW} = 2.5\cdot in$

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g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $wgt_r = 66.375\cdot psf$ Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $wgt_w = 39.333\cdot psf$ Wall Dead Load

$wgt_{flr}^T = (41.15 \quad 40.4 \quad 39.84 \quad 39.42 \quad 39.08 \quad 38.8 \quad 38.58 \quad 37.78 \quad 37.66)\cdot psf$ Floor Dead Load

$W_{lat} = 45.986\cdot psf$ Wind Load

$w_{llf.0} = 250\cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200\cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150\cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$A_r := L_r\cdot W_r$ $A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\,ft^2$ Area of Roof/Floor

$A_f := L_f\cdot W_f$ $A_f^T = (123.7 \quad 145.7 \quad 167.6 \quad 189.5 \quad 211.4 \quad 233.3 \quad 255.2 \quad 277.2 \quad 299.1)\,ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$ Allowable Resisting Moment

$M_R^T = (539.9 \quad 553.6 \quad 567.3 \quad 581 \quad 594.7 \quad 608.5 \quad 622.2 \quad 635.9 \quad 649.6)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$ Overturning Moment due to Wind

$M_{OT.wind}^T = (48.9 \quad 57.1 \quad 65.2 \quad 73.4 \quad 81.5 \quad 89.7 \quad 97.8 \quad 106 \quad 114.1)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^T = (73.4 \quad 85.6 \quad 97.8 \quad 110 \quad 122.3 \quad 134.5 \quad 146.7 \quad 159 \quad 171.2)\cdot kip\cdot ft$

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\begin{bmatrix} L_r\cdot W_r\cdot(wgt_r)\cdot\max(H_E) + L_r\cdot W_r\cdot(0.25w_{llr})\cdot\max(H_E) \ldots \\ +\left(2L_f\cdot\max(H_E) + 2W_{ew}\cdot\max(H_E)\right)\cdot\dfrac{wgt_w\cdot\max(H_E)}{2} \ldots \\ +L_f\cdot W_f\cdot wgt_{flr}\cdot\dfrac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot(0.25w_{llf.0})\cdot t_{floor} \end{bmatrix}$$

Overturning Moment due to seismic

$$M_{OT.seismic}^{T} = (39.347 \quad 43.94 \quad 48.533 \quad 53.126 \quad 57.718 \quad 62.311 \quad 66.904 \quad 71.497 \quad 76.09)\cdot kip\cdot ft$$

$$1.5\,M_{OT.seismic}^{T} = (59.02 \quad 65.91 \quad 72.799 \quad 79.688 \quad 86.578 \quad 93.467 \quad 100.356 \quad 107.246 \quad 114.135)\cdot kip\cdot ft$$

$$M_{ot.w.net} := \left(1.5M_{OT.wind} - M_R\right) \qquad M_{ot.s.net} := \left(1.5M_{OT.seismic} - M_R\right)$$

$$M_{ot.w.net}^{T} = (-466.5 \quad -468 \quad -469.5 \quad -471 \quad -472.5 \quad -474 \quad -475.5 \quad -477 \quad -478.4)\cdot kip\cdot ft$$

$$M_{ot.s.net}^{T} = (-480.9 \quad -487.7 \quad -494.5 \quad -501.3 \quad -508.2 \quad -515 \quad -521.8 \quad -528.7 \quad -535.5)\cdot kip\cdot ft$$

$chk_{S7.15} := \text{if}\left(\max\left(M_{ot.w.net}, M_{ot.s.net}\right) \geq 0, \text{"NG"}, \text{"OK"}\right)$ $chk_{S7.15} = \text{"OK"}$

Sliding

$$WT := \begin{pmatrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ +2W_{ew}\cdot\min(H_E)\cdot wgt_w + 2L_f\cdot W_f\cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^{T} = (170.991 \quad 173.343 \quad 175.695 \quad 178.047 \quad 180.399 \quad 182.75 \quad 185.102 \quad 187.454 \quad 189.806)\cdot kip$$

$$WT_{max} := \max\begin{bmatrix} A_r\cdot(wgt_r + w_{misc.r}) + A_f\cdot(wgt_{flr} + w_{misc.f}) \ldots \\ +2L_f\cdot\max(H_E)\cdot(wgt_w + w_{misc.w}) \ldots \\ +2\cdot W_{ew}\cdot\max(H_E)\cdot wgt_w + \left[A_r\cdot w_{llr} + A_r\cdot(w_{llf.0})\right] \end{bmatrix}$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 263.948\cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in\cdot 4in$ $A_{pad} = 16\cdot in^2$

$S_{bp} := 3ft$ Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad} = 7.3\cdot kip$ Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 456.3\cdot psi$ Compressive Stress per Pad

$F_{pad} := 800psi$ Allowable Stress per pad

$F_P := 0.7\cdot 0.85f'_{c.fdn}$ $F_P = 1785.1\cdot psi$ Allowable Stress of Fdn Concrete

$chk_{S7.16} := \text{if}\left(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, \text{"OK"}, \text{"NG"}\right)$ $chk_{S7.16} = \text{"OK"}$

$$wt_{pad.big} := \max\left[\frac{\max\left(WT_{max}\right)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad.big} = 10.152\cdot kip$ Weight per Bearing Pad

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$ $f_{pad.big} = 634.5 \cdot psi$ Compressive Stress per Pad

$chk_{S7.17} := if\left(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, "OK", "NG"\right)$ $chk_{S7.17} = "OK"$

$\mu := 0.25$ Coefficient of Friction

Wind $V_{net.w.sl} := max\left[0kip, \left[\begin{matrix}\mu \cdot \left[-(WT) + 1.5\left|P_{wind_3}\right| \cdot \left(A_r\right)\right] \ldots \\ + 1.5 W_{lat} L \cdot max\left(H_E\right)\end{matrix}\right]\right]$ $V_{net.w.sl} = 0 \cdot kip$ Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := max\left[0kip, 1.5 \cdot \left[0.7 C_s \cdot \left(\begin{matrix}0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT\end{matrix}\right) + -\mu \cdot \left(\begin{matrix}0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT\end{matrix}\right)\right]\right]$

$V_{net.s.sl} = 0 \cdot kip$ Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$chk_{S7.18} := if\left(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, "Slide Plates Req'd", "OK"\right)$ $chk_{S7.18} = "OK"$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \left\| \begin{matrix} max\left(2, ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) & if \ \frac{ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ max\left[2, \left(ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] & otherwise \end{matrix}\right.$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

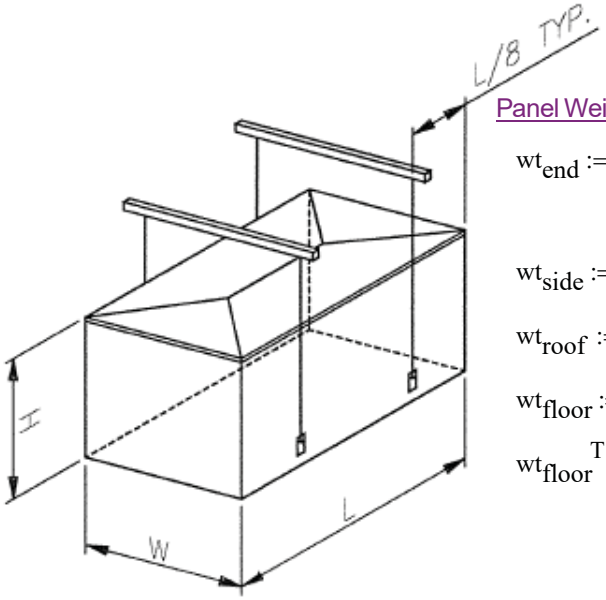
$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (10 \ \ 12 \ \ 14 \ \ 16 \ \ 18 \ \ 20 \ \ 22 \ \ 24 \ \ 26) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (10 \ \ 12 \ \ 14 \ \ 16 \ \ 18 \ \ 20 \ \ 22 \ \ 24 \ \ 26) \, ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3 \, ft$ on center.

8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5191 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 445 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 797 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (480 \quad 471 \quad 465 \quad 460 \quad 456 \quad 453 \quad 450 \quad 441 \quad 439) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) $M/(0.9f_y d)$			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	12.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.016	0.042
12.00	14.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.029	0.016
12.00	16.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.044	0.011
12.00	18.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.062	0.038
12.00	20.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.082	0.065
12.00	22.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.105	0.092
12.00	24.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.131	0.119
12.00	26.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.159	0.146
12.00	28.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.189	0.172
12.00	9.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.000	0.009
12.00	10.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.005	0.015
12.00	10.67	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.008	0.020
12.00	18.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.058	0.029
12.00	20.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.078	0.050
12.00	22.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.100	0.071
12.00	24.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.124	0.092
12.00	26.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.150	0.113
12.00	28.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.179	0.133
12.00	12.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.014	0.029
12.00	14.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.026	0.011
12.00	16.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.040	0.008
12.00	18.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.056	0.026
12.00	20.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.075	0.044
12.00	22.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.096	0.062
12.00	24.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.120	0.080
12.00	26.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.145	0.099
12.00	28.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.173	0.117

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b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

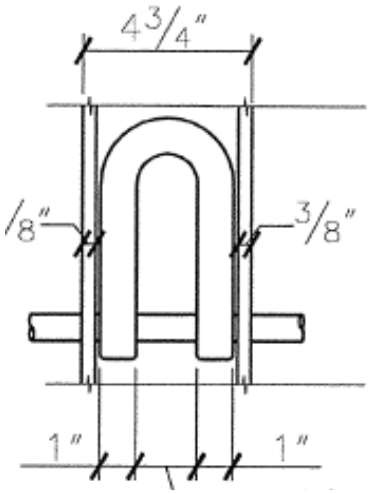
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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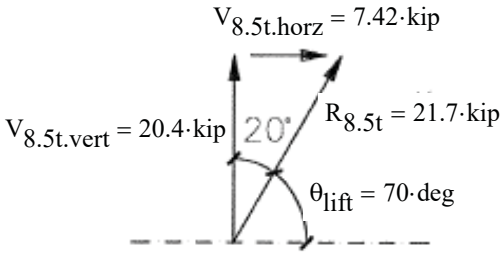


$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

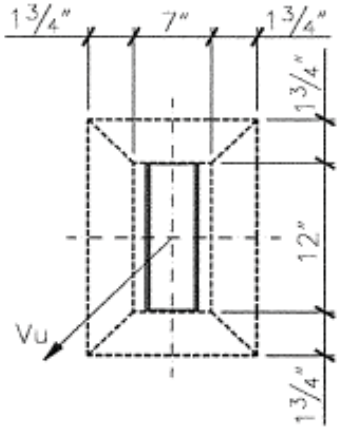
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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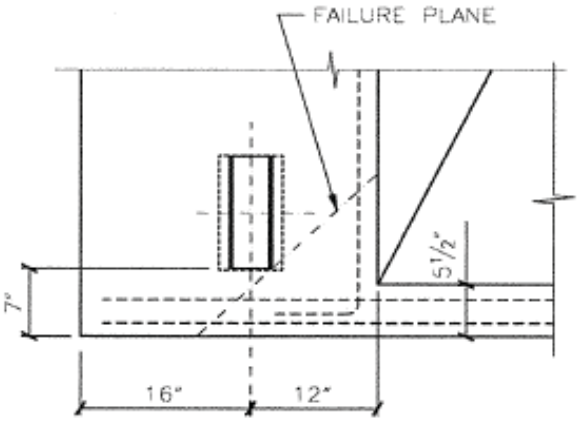
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

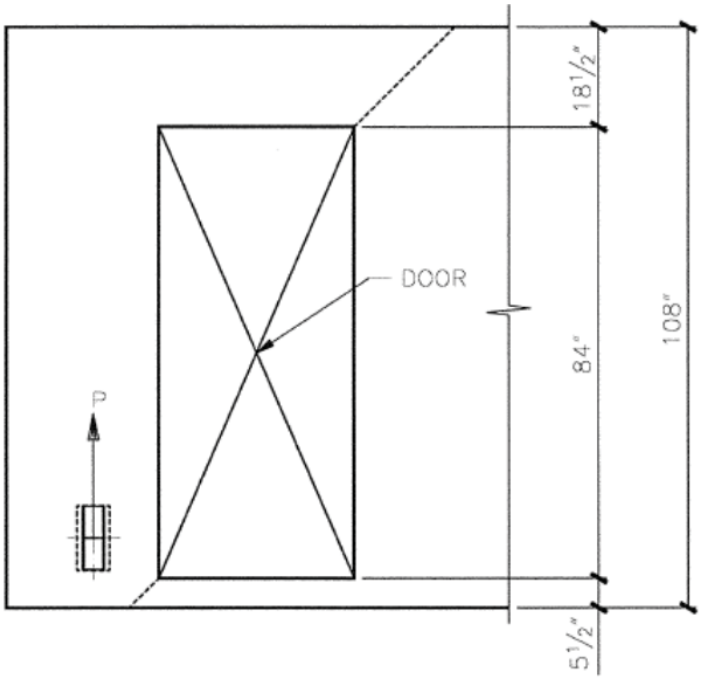
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

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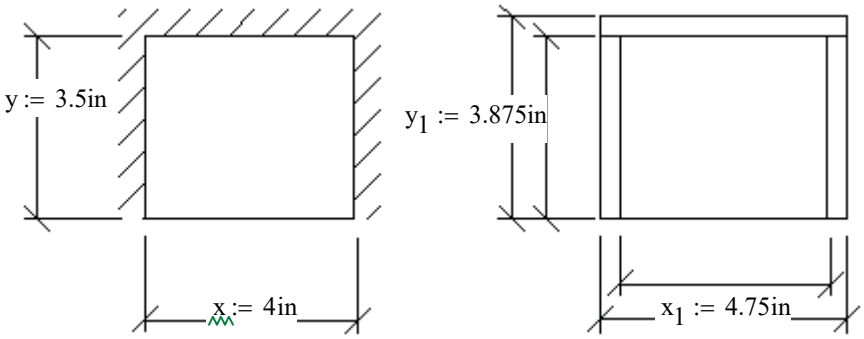
Bearing Pressure

Vertical



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$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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Manufacturer: Modular Connections, LLC

$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

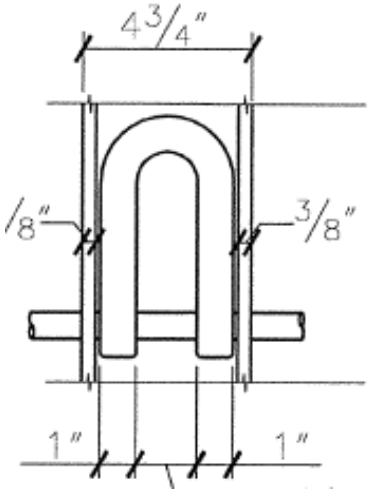
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

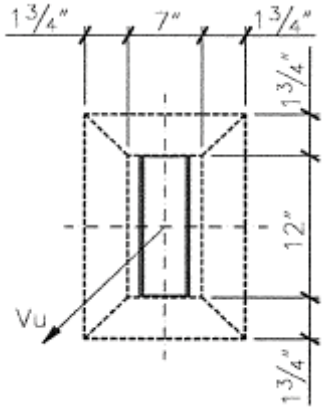
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

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Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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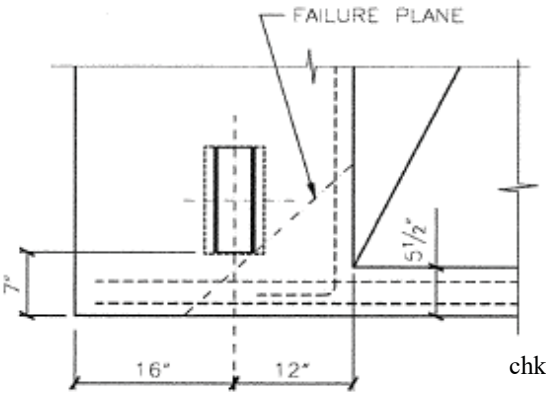
Manufacturer:	Modular Connections, LLC

Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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Plan No.:	MET-4193-Standard Concrete
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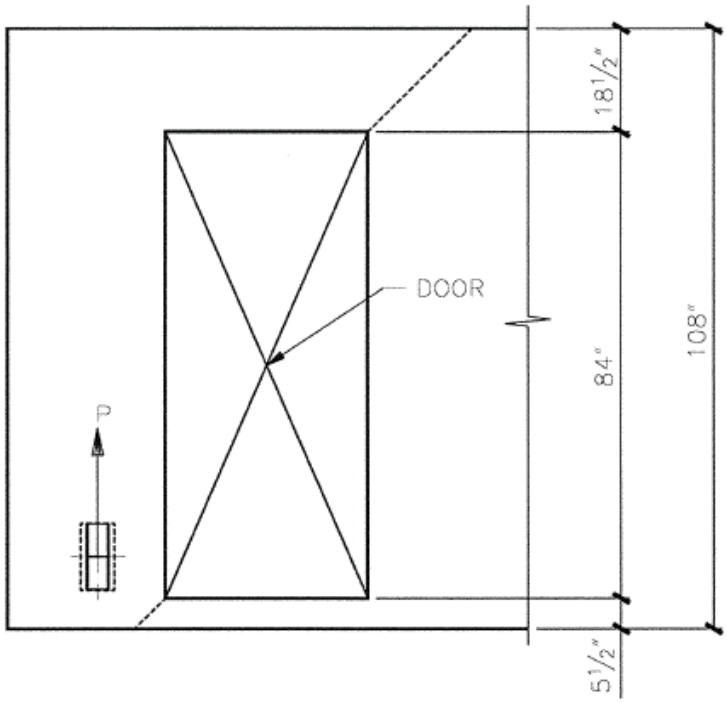


$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

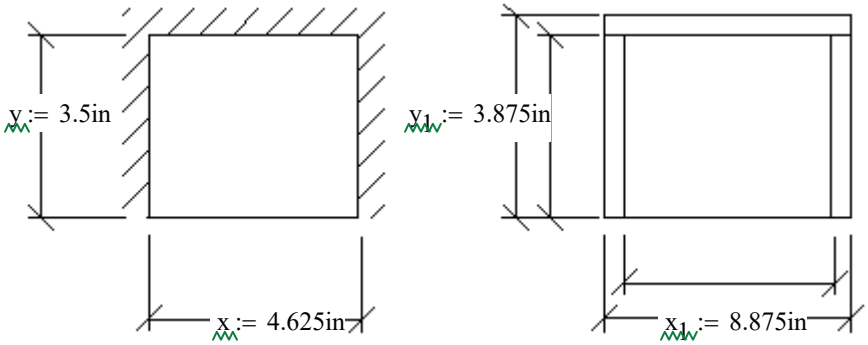
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

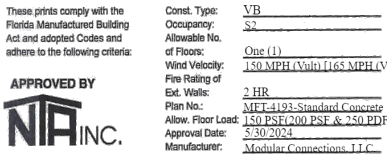
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



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Allowable No. of Floors: One (1)
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 22.3 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 13.4 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2\max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 24.9 \cdot kip$ Wall Dead Load

$w_{ew} := 2\max(wt_{end})$ $w_{ew} = 10.4 \cdot kip$ Weight of End Wall

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Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 71 \cdot kip$ Factored Dead Load of Shelter

$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}}$ $CG = 7.3\,ft$ Center of gravity from bottom of shelter

Tie Down Bracket $Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 129.3 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}}$ $F_{turn} = 30.3 \cdot kip$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 231.2 \cdot kip \cdot ft$ Overtruning Moment

$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2}$ $M_{OT.Resist} = 271.7 \cdot kip \cdot ft$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -40.5 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}}$ $F_{OT.trans} = -4.8 \cdot kip$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans} DL$ $F_{net.slide.trans} = 16 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 8 \cdot kip$ Maximum Shear Force Per Tie-Down, θ_{min}

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 8 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$

$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right)$ $\phi P_{pl.FlrTie} = 26.928 \cdot kip$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.298$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.698$ < 1.0

$$\max\left(\frac{T_{tie.max}}{\phi P_{pl.FlrTie}}\right) + \frac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.996$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 0.996$ < 1.2

$chk_{8.Flr.Tie}$ = "OK"

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9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 6 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 2.7 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 22.12 \quad U_{R3.25} = 0.045$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2)\cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 6 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 2.7 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 27.12 \quad U_{R4.75} = 0.037$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (11.292 \quad 13.292 \quad 15.292 \quad 17.292 \quad 19.292 \quad 21.292 \quad 23.292 \quad 25.292 \quad 27.292)\,ft$ Length of Floor

$wgt_{flr}^T = (41.147 \quad 40.396 \quad 39.841 \quad 39.415 \quad 39.077 \quad 38.803 \quad 38.575 \quad 37.781 \quad 37.662)\cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 4.184\cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 3.816\cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.883 \\ 0.68 \\ 14.69 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 17.253 \quad U_F = 0.058$

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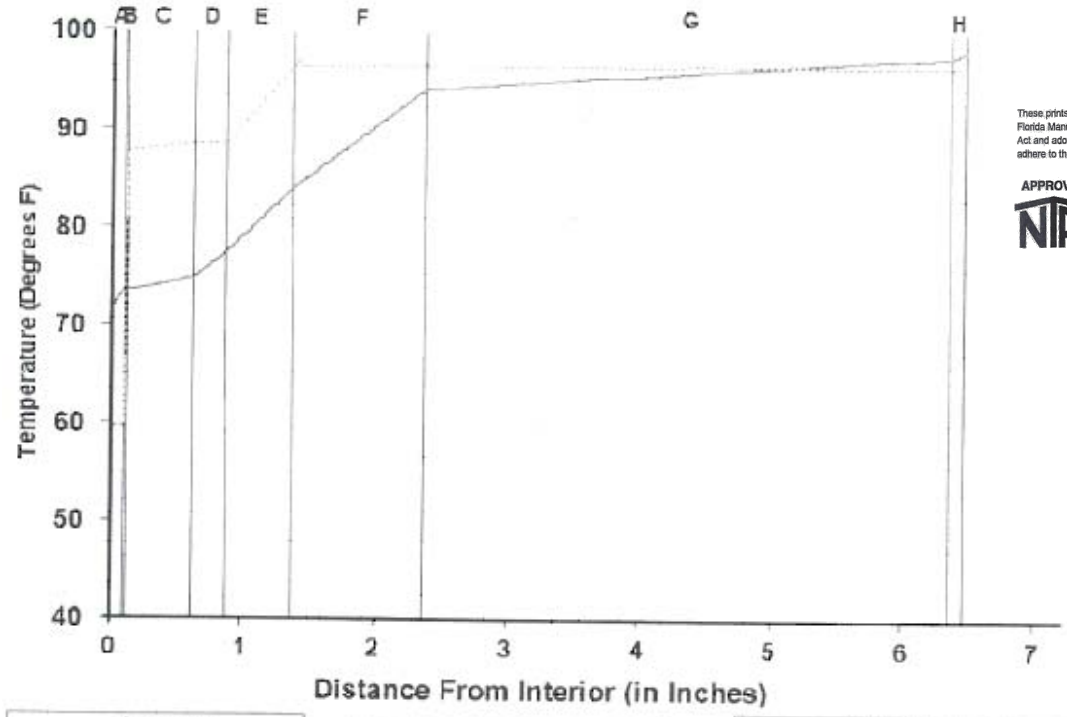
Dewpoint Analysis - Dow Chemical

Sam Cain



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Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

May 17 2006 12:16:28 PM

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10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

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Manufacturer:	Modular Connections, LLC

$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 65.753 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 935.2 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 1987 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$ Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.0101$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.242 \cdot in^2$ Req'd Area of Steel per foot Width

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Manufacturer:	Modular Connections, LLC

Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 351 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG")$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 7.516\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 15.032\,ft$

Maximum allowable opening width

$w_{d.max} := \max(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4})$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

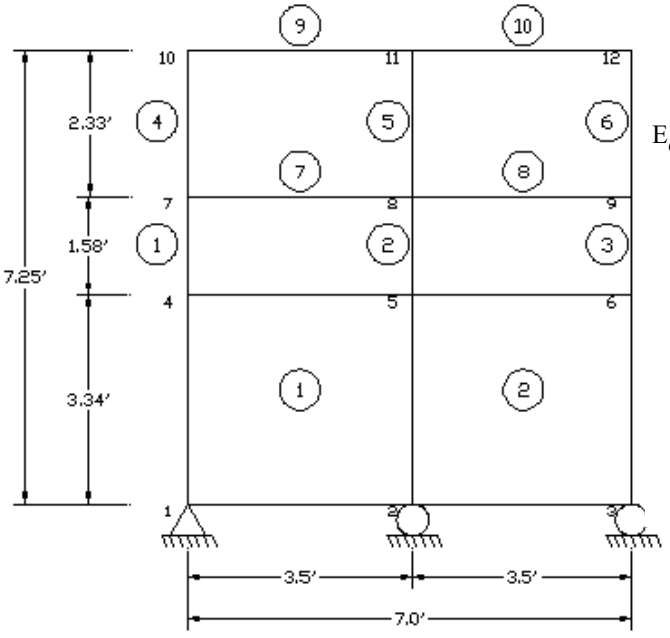
$chk_{opn.width} := if(w_d \geq w_{d.max}, "OK", "NG")$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.523 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.523 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.523 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2})$ In-Plane Force at Endwall



$V_u = 4.523 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in^2)	I (in^4)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.523 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.569 \cdot kip \cdot ft$

Shear

$\phi Vc_{6d.opn} := \phi_V \cdot 0.85 \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot \left(d_7 - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00115$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.145 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$ Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PSF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.com

APPENDIX "B"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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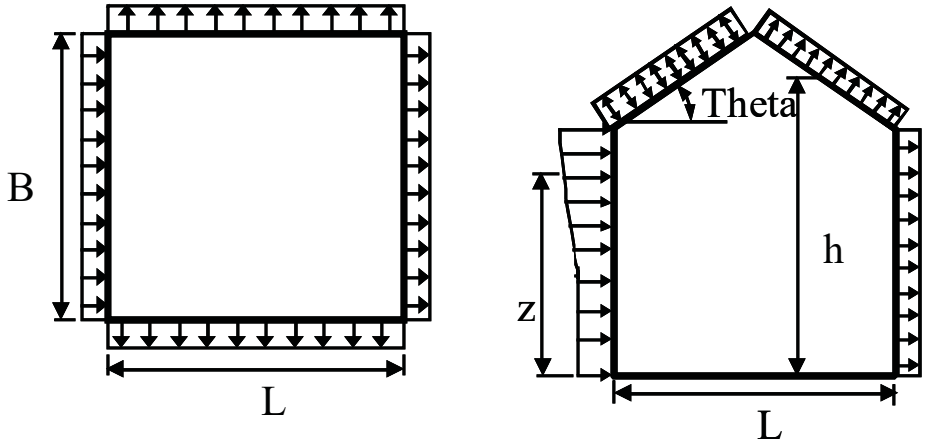
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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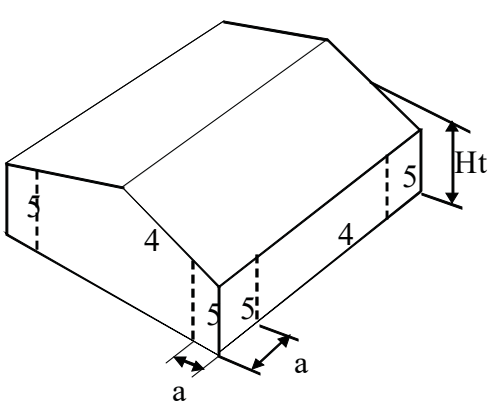
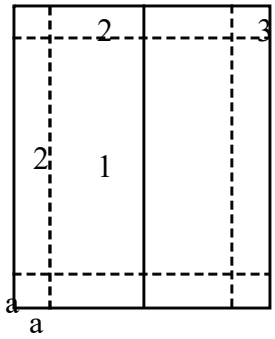
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTION, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Manufacturer: Modular Connections, LLC

Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

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Fire Rating of Ext. Walls:	2 HR
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APPENDIX "C"

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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

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MODULAR CONNECTIONS, LLC

Cover =

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

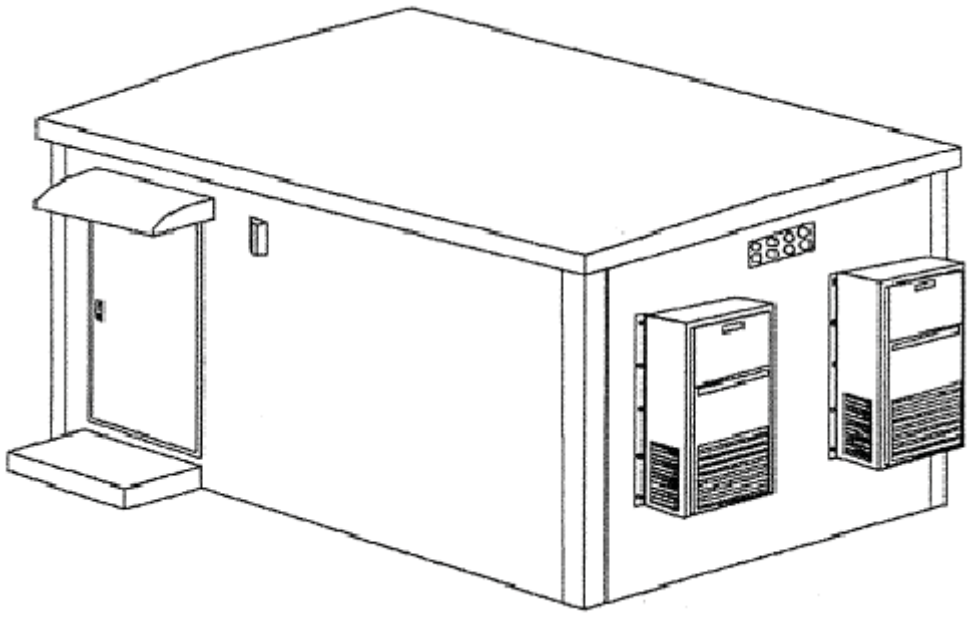
" IBC 2021 (ASCE 7-16) "

" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #5"

" REVISION - 0: DATE MARCH 11, 2024 "

""



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" copied or traced in whole or part without our written consent. "

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" WALL DESIGN ---------- S3.1 - S3.16 "
" FLOOR DESIGN ---------- S4.1 - S4.31 "
" LATERAL DESIGN ---------- S5.1 - S5.24 "
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" FIRE RATING & THERMAL EFFICIENCY---------- S9.1 - S9.3 "
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""

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" APPENDIX A - BULLET RESISTANCE TEST "
" APPENDIX B - AWINING / HOOD ATTACHMENT DESIGN "
" APPENDIX C - DEVELOPMENT OF REINFORCEMENT "

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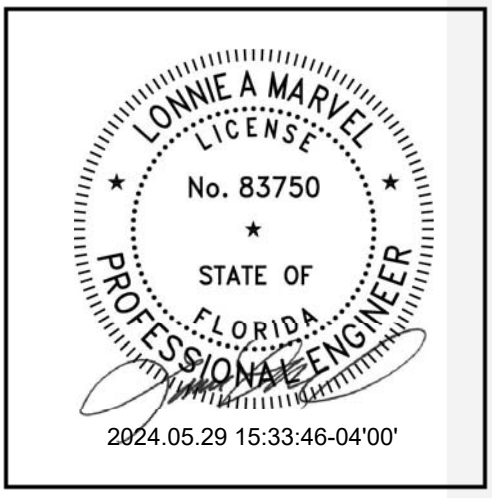
I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.



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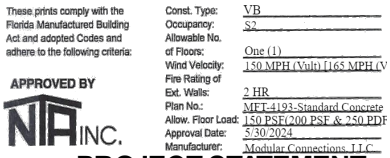

PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 39.2 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 56.5 psf"
"Roof Live Load ---------- 80 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 150 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 35.8 psf"
"Uplift Wind Pressure, MWFRS ---------- 53.5 psf"
"Zone 4 Wind Pressure, C & C ---------- 53.3 psf"
"Zone 5 Wind Pressure, C & C ---------- 65.8 psf"
"Zone 1 Wind Pressure, C & C ---------- 78.2 psf"
"Zone 2 Wind Pressure, C & C ---------- 103.2 psf"
"Zone 3 Wind Pressure, C & C ---------- 140.7 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 32 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 39.5 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.8"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.2"

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1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$)
Welding electrodes : E70XX
Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"
$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"
$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"
$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 80 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"
$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot psf \qquad q_h = 41.616 \cdot psf \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 20.81 & 35.79 \\ -25.18 & -10.2 \\ -32.25 & -17.27 \\ -53.48 & -38.49 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 35.79 & 20.81 \\ -10.2 & -25.18 \\ -17.27 & -32.25 \\ -38.49 & -53.48 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 35.79 \\ 25.18 \\ 32.25 \\ -53.48 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 45.99 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 150 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 41.616 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -53.3 & -38.3 \\ 34.1 & 49.1 \\ -65.8 & -50.8 \\ 5 & 20 \\ -78.2 & -63.3 \\ 5 & 20 \\ -103.2 & -88.2 \\ 5 & 20 \\ -140.7 & -125.7 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -38.3 & -53.3 \\ 49.1 & 34.1 \\ -50.8 & -65.8 \\ 20 & 5 \\ -63.3 & -78.2 \\ 20 & 5 \\ -88.2 & -103.2 \\ 20 & 5 \\ -125.7 & -140.7 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 53.3 \\ 53.3 \\ 65.8 \\ 78.2 \\ 103.2 \\ 140.7 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 53.3 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 65.8 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 78.2 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 103.2 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 140.7 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 107.369 \cdot psf$$

Max Roof Uplift Pressure (average)

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 1.16 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1.036 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 1.202$ $S_{MS.ref} = \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"}$

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.801$ $S_{DS.ref} = \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"}$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} = \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"}$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} = \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"}$

$$H_E = \begin{pmatrix} 9.563 \\ 10.563 \\ 11.229 \end{pmatrix} \text{ft} \qquad H_{E.ref} = \text{"Exterior Height of Building"}$$

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right] \qquad Ta = 0.104 \quad Ta_{ref} = \text{"Approx Per., Eqn (12.8-7)"}$$

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_0 = 0.209$ $T_s := \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_s = 1.047$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.2003$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E}\right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E}\right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1}\right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0353$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0\right)\right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max}\right), C_{s.min}\right)$ $C_s = 0.2$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 5 \cdot in$ Thickness @ ridge $t_{r.r} = 6.5 \cdot in$ $t_{r.a} = 5.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.1404 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 56.54 \cdot psf$ $DL_{r.mn.ref} :=$ "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 63.54 \cdot psf$ $DL_{r.mx.ref} :=$ "Max. Roof DL"

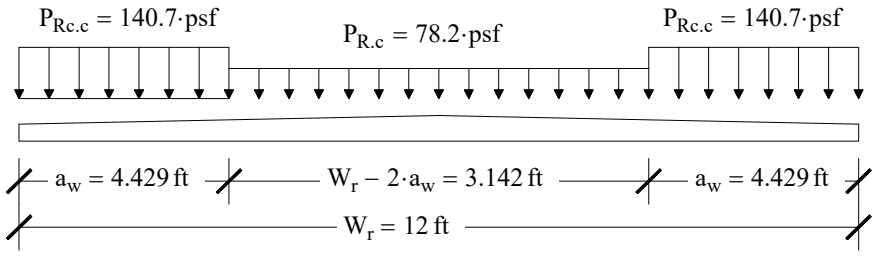
$w_{llr} = 80 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 140.7 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 78.2 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.429\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -24.247 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -112.255 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 10.182 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -10.182 \cdot psf$ Minimum Upward Roof Seismic Load

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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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$$\text{LRFD} = \begin{pmatrix} \text{"1a"} & \text{"1.4(D+F)"} \\ \text{"2a"} & \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"3a"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"3b+"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"3b-"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"4a+"} & \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"4a-"} & \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"5a+"} & \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"5a-"} & \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"6a+"} & \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"6a-"} & \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"7a+"} & \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"7a-"} & \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Wu_{S2.0} = \begin{pmatrix} 88.96 \\ 116.25 \\ 204.25 \\ 204.25 \\ 148.12 \\ 116.25 \\ 60.12 \\ 92.78 \\ 72.42 \\ 50.89 \\ -61.37 \\ 61.07 \\ 40.71 \end{pmatrix} \cdot psf$$

Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref} = \text{"Max Span of Roof Panel"}$

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 204.25 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -61.367 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

	$Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$	$Vu_{ref} = \text{"Ultimate Shear"}$
Roof Live	$Mu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}$	$Vu_{2.1.mx} := \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}$
Load	$Mu_{2.1.mx} = 37.071 \cdot kip \cdot in$	$Vu_{2.1.mx} = 1.123 \cdot kip$
Option 1	$Mu_{2.1.mn} := \min\left(0kip \cdot in, \lvert w_{u.mn} \rvert \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$	$Vu_{2.1.mn} := \min\left(0kip, \lvert w_{u.mx} \rvert \cdot b_{strip} \cdot \frac{l_r}{2}\right)$
	$Mu_{2.1.mn} = 0 \cdot kip \cdot in$	$Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 80 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$

$B_{2.1} = \text{"#4"}$ $B_{2.1.ref} = \text{"Transverse Rebar Size"}$ $S_{2.1} = 12 \cdot in$ $S_{2.1.ref} = \text{"Transverse Rebar Spacing"}$

$B_{2.2} = \text{"#4"}$ $B_{2.2.ref} = \text{"Longitudinal Rebar Size"}$ $S_{2.2} = 12 \cdot in$ $S_{2.2.ref} = \text{"Longitudinal Rebar Spacing"}$

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref} = \text{"Clear Cover, From Bot. w/ Keyway"}$

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref} = \text{"Clear Cover, From Bot. w/o Keyway"}$

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref} = \text{"Roof Width Bar Diam."}$ $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref} = \text{"Roof Width Bar Area."}$

$As_{2.1} = 0.2 \cdot in^2$ $As_{2.1.ref} = \text{"Roof Width Area Reinf."}$ $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref} = \text{"Roof Length Bar Diam."}$

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref} = \text{"Roof Length Bar Area."}$ $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref} = \text{"Roof Length Area Reinf."}$

$A_{s.mim.2} = 0.1404 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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Fire Rating of Ext. Walls: 2 HR
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer: Modular Connections, LLC

$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 4 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.2 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.0042$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.261 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 46.4 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.317 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.03487$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 41.8 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 37.1 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 2.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 3.523 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 13.685 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 3.523 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 2.642 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 1.123 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{F.2.3} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.5} = 0.261 \cdot in$ a_{ref} = "Depth of Comp. Block, Eqn (22.2.2.4.1)"

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $M_{n.2.3} = 19.4 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $c_{2.3} = 0.317 \cdot in$ c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\varepsilon_{2.3} = 0.01357$ ε_{ref} = "Tensile Strain, Extreme Layer"

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ ϕ_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\phi M_{n.2.3} = 17.5 \cdot kip \cdot in$ $\phi M_{n.ref}$ = "Flexural Strength, -ve, Sect (22.3)"

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $Mu_{n.ref}$ = "Ultimate Moment, -ve"

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.5}$ = "OK"

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $d_{V.2.4} = 1.75 \cdot in$ d_{ref} = "Depth of Reinf."

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\lambda_{s.2.4} = 1$ $\lambda_{s.ref}$ = "Size Effect Fact, Eqn (22.5.5.1.3)"

$\rho_{2.3} = 0.0095$ ρ_{ref} = "Reinf. Ratio"

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.a} = 2.031 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $Vc_{2.4} = 2.031 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\phi Vc_{2.4} = 1.523 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Vu_{2.1.mn} = 0 \cdot kip$ Vu_{ref} = "Ultimate Shear"

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $chk_{2.6}$ = "OK"

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 190.109 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 2.875 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 28.28 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 26.053 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 11.533 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 95.055 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -6.654 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 47.527 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 47.53 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 190.1 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 78.3 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.03681 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.202 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.16519 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.441\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.083\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.524\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 251.9$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 80 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 80 \cdot psf$

$P_{W.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_0}$ = " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 53.3 \cdot psf$ $GC_{p.cc.max.ref_1}$ = " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 65.8 \cdot psf$ $GC_{p.cc.max.ref_2}$ = "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 53.27 \\ 65.75 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 80 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.1 \\ 1.19 \\ 1.72 \\ 1.72 \\ 2.04 \\ 1.19 \\ 1.51 \\ 1.05 \\ 0.92 \\ 0.62 \\ 1.27 \\ 0.69 \\ 0.56 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 26.6 \\ 26.6 \\ 53.3 \\ 26.6 \\ 9.1 \\ -9.1 \\ 53.3 \\ 53.3 \\ 9.1 \\ -9.1 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))\ " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 2.04 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.54 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.56 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.09 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.47 \cdot kip$

$P_{u.3b.h1}{}^T = (1.105 \quad 1.187 \quad 1.715 \quad 1.715 \quad 2.037 \quad 1.187 \quad 1.509 \quad 1.046 \quad 0.924 \quad 0.624 \quad 1.268 \quad 0.685 \quad 0.563) \cdot kip$

$M_{u.3b.h1}{}^T = (0 \quad 0 \quad 0 \quad 0.27 \quad 0.27 \quad 0.539 \quad 0.27 \quad 0.092 \quad -0.092 \quad 0.539 \quad 0.539 \quad 0.092 \quad -0.092) \cdot kip \cdot ft$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.12 \;\; 0.12 \;\; 0.24 \;\; 0.12 \;\; 0.041 \;\; -0.041 \;\; 0.24 \;\; 0.24 \;\; 0.041 \;\; -0.041)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10ft$ & Roof Live Load $w_{llr} = 80\cdot psf$

$Pu_{3b.h2}^{T} = (1.17 \;\; 1.24 \;\; 1.77 \;\; 1.77 \;\; 2.09 \;\; 1.24 \;\; 1.56 \;\; 1.1 \;\; 0.98 \;\; 0.66 \;\; 1.3 \;\; 0.72 \;\; 0.6)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 9.1 \;\; -9.1 \;\; 53.3 \;\; 53.3 \;\; 9.1 \;\; -9.1)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2})$ $\quad P_{u.3b.h2.mx} = 2.09\cdot kip$ $\quad M_{u.3b.h2.mx} := \max(M_{u.3b.h2})$ $\quad M_{u.3b.h2.mx} = 0.67\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2})$ $\quad P_{u.3b.h2.mn} = 0.6\cdot kip$ $\quad M_{u.3b.h2.mn} := \min(M_{u.3b.h2})$ $\quad M_{u.3b.h2.mn} = -0.11\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|)$ $\quad V_{u.3b.h2.mx} = 0.52\cdot kip$

$P_{u.3b.h2}^{T} = (1.168 \;\; 1.242 \;\; 1.77 \;\; 1.77 \;\; 2.092 \;\; 1.242 \;\; 1.564 \;\; 1.101 \;\; 0.979 \;\; 0.659 \;\; 1.304 \;\; 0.72 \;\; 0.598)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.333 \;\; 0.333 \;\; 0.666 \;\; 0.333 \;\; 0.113 \;\; -0.113 \;\; 0.666 \;\; 0.666 \;\; 0.113 \;\; -0.113)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.133 \;\; 0.133 \;\; 0.266 \;\; 0.133 \;\; 0.045 \;\; -0.045 \;\; 0.266 \;\; 0.266 \;\; 0.045 \;\; -0.045)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667ft$ & Roof Live Load $w_{llr} = 80\cdot psf$

$Pu_{3b.h3}^{T} = (1.21 \;\; 1.28 \;\; 1.81 \;\; 1.81 \;\; 2.13 \;\; 1.28 \;\; 1.6 \;\; 1.14 \;\; 1.01 \;\; 0.68 \;\; 1.33 \;\; 0.74 \;\; 0.62)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 26.6 \;\; 26.6 \;\; 53.3 \;\; 26.6 \;\; 9.1 \;\; -9.1 \;\; 53.3 \;\; 53.3 \;\; 9.1 \;\; -9.1)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3})$ $\quad P_{u.3b.h3.mx} = 2.13\cdot kip$ $\quad M_{u.3b.h3.mx} := \max(M_{u.3b.h3})$ $\quad M_{u.3b.h3.mx} = 0.76\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3})$ $\quad P_{u.3b.h3.mn} = 0.62\cdot kip$ $\quad M_{u.3b.h3.mn} := \min(M_{u.3b.h3})$ $\quad M_{u.3b.h3.mn} = -0.13\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|)$ $\quad V_{u.3b.h3.mx} = 0.56\cdot kip$

$P_{u.3b.h3}^{T} = (1.211 \;\; 1.278 \;\; 1.806 \;\; 1.806 \;\; 2.128 \;\; 1.278 \;\; 1.6 \;\; 1.137 \;\; 1.015 \;\; 0.683 \;\; 1.327 \;\; 0.744 \;\; 0.622)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.379 \;\; 0.379 \;\; 0.758 \;\; 0.379 \;\; 0.129 \;\; -0.129 \;\; 0.758 \;\; 0.758 \;\; 0.129 \;\; -0.129)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.142 \;\; 0.142 \;\; 0.284 \;\; 0.142 \;\; 0.048 \;\; -0.048 \;\; 0.284 \;\; 0.284 \;\; 0.048 \;\; -0.048)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\quad S_{3.0}$ = "N/A" $\cdot in \quad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \quad d_{3b} := \frac{t_w}{2} - 0.25in \quad d_{3b} = 1.75\cdot in$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.559 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.076$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.929$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

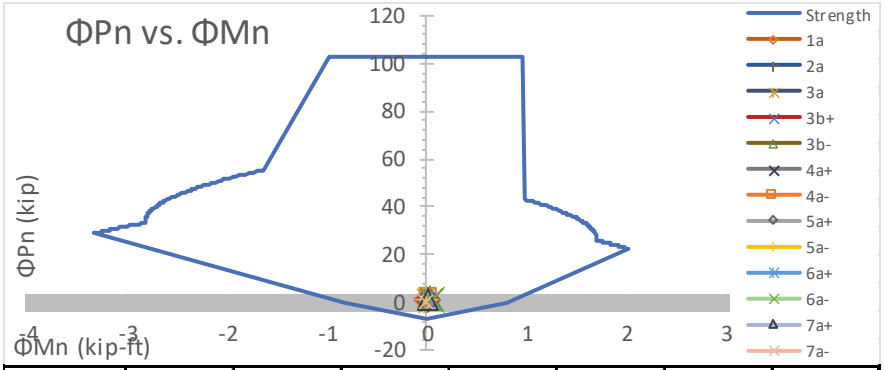
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.098$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.91$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.116$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.896$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.896$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.2%	1.2%	1.8%	2.94%	2.9%	6.1%	3.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.1%	1.0%	6.3%	6.0%	1.1%	1.1%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref}$ = "Min. Space to Adjacent Opening"

$b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref}$ = "Strip Width Btwn Openings"

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1}$ = "#6" $B_{3.1.ref}$ = "Wall Vert. Perim. Reinf." $B_{3.2}$ = "#4" $B_{3.2.ref}$ = "Wall Top. Perim. Reinf."

$N_{B.3.1} = 1$ $N_{B.3.1.ref}$ = "# Wall Vert. Perim. Bars" $N_{B.3.2} = 2$ $N_{B.3.2.ref}$ = "# Wall Top Perim. Bars"

$D_{3c.1} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.2} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.1} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.2} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.3}$ = "#6" $B_{3.3.ref}$ = "EW Bot Perim. Reinf." $B_{3.4}$ = "#4" $B_{3.4.ref}$ = "SW Bot Perim. Reinf."

$N_{B.3.3} = 1$ $N_{B.3.3.ref}$ = "# EW Bot Perim. Bars" $N_{B.3.4} = 2$ $N_{B.3.4.ref}$ = "# SW Bot Perim. Bars"

$D_{3c.3} = 0.75 \cdot in$ D_{ref} = "Bar Diam." $D_{3c.4} = 0.5 \cdot in$ D_{ref} = "Bar Diam."

$A_{3c.3} = 0.44 \cdot in^2$ A_{ref} = "Area of Reinf." $A_{3c.4} = 0.4 \cdot in^2$ A_{ref} = "Area of Reinf."

$B_{3.5}$ = "#5" $B_{3.5.ref}$ = "Wall Opening Reinf." $D_{3c.5} = 0.625 \cdot in$ D_{ref} = "Bar Diam."

$N_{B.3.5} = 2$ $N_{B.3.5.ref}$ = "# Wall Opening Bars" $A_{3c.5} = 0.62 \cdot in^2$ A_{ref} = "Area of Reinf."

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 80 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}^T = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 36.3 \;\; -36.3 \;\; 213.1 \;\; 213.1 \;\; 36.3 \;\; -36.3) \cdot plf$

$P_{u.3c.h1d1}^T = (4.42 \;\; 4.75 \;\; 6.86 \;\; 6.86 \;\; 8.15 \;\; 4.75 \;\; 6.04 \;\; 4.19 \;\; 3.7 \;\; 2.5 \;\; 5.07 \;\; 2.74 \;\; 2.25) \cdot kip$

$M_{u.3c.h1d1}^T = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.22 \;\; -0.22 \;\; 1.31 \;\; 1.31 \;\; 0.22 \;\; -0.22) \cdot kip \cdot ft$

$V_{u.3c.h1d1}^T = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.13 \;\; -0.13 \;\; 0.75 \;\; 0.75 \;\; 0.13 \;\; -0.13) \cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d1.mx} = 8.15 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.31 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.47 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.25 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.22 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 80 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$W_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 27.2 \;\; -27.2 \;\; 159.8 \;\; 159.8 \;\; 27.2 \;\; -27.2)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.31 \;\; 3.56 \;\; 5.15 \;\; 5.15 \;\; 6.11 \;\; 3.56 \;\; 4.53 \;\; 3.14 \;\; 2.77 \;\; 1.87 \;\; 3.8 \;\; 2.06 \;\; 1.69)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.98 \;\; 0.98 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 6.11\cdot kip$ $M_{u.3c.h1d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.1\cdot kip$

$P_{u.3c.h1d2.mn} = 1.69\cdot kip$ $M_{u.3c.h1d2.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 22.7 \;\; -22.7 \;\; 133.2 \;\; 133.2 \;\; 22.7 \;\; -22.7)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.76 \;\; 2.97 \;\; 4.29 \;\; 4.29 \;\; 5.09 \;\; 2.97 \;\; 3.77 \;\; 2.62 \;\; 2.31 \;\; 1.56 \;\; 3.17 \;\; 1.71 \;\; 1.41)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.14 \;\; -0.14 \;\; 0.82 \;\; 0.82 \;\; 0.14 \;\; -0.14)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.08 \;\; -0.08 \;\; 0.47 \;\; 0.47 \;\; 0.08 \;\; -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 5.09\cdot kip$ $M_{u.3c.h1d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 0.92\cdot kip$

$P_{u.3c.h1d3.mn} = 1.41\cdot kip$ $M_{u.3c.h1d3.mn} = -0.14\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 36.3 \;\; -36.3 \;\; 213.1 \;\; 213.1 \;\; 36.3 \;\; -36.3)\cdot plf$

$P_{u.3c.h2d1}^{T} = (4.67 \;\; 4.97 \;\; 7.08 \;\; 7.08 \;\; 8.37 \;\; 4.97 \;\; 6.25 \;\; 4.4 \;\; 3.91 \;\; 2.64 \;\; 5.21 \;\; 2.88 \;\; 2.39)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.22 \;\; -0.22 \;\; 1.31 \;\; 1.31 \;\; 0.22 \;\; -0.22)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.13 \;\; -0.13 \;\; 0.75 \;\; 0.75 \;\; 0.13 \;\; -0.13)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 8.37\cdot kip$ $M_{u.3c.h2d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.47\cdot kip$

$P_{u.3c.h2d1.mn} = 2.39\cdot kip$ $M_{u.3c.h2d1.mn} = -0.22\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 27.2 \;\; -27.2 \;\; 159.8 \;\; 159.8 \;\; 27.2 \;\; -27.2)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.51 \;\; 3.72 \;\; 5.31 \;\; 5.31 \;\; 6.27 \;\; 3.72 \;\; 4.69 \;\; 3.3 \;\; 2.94 \;\; 1.98 \;\; 3.91 \;\; 2.16 \;\; 1.79)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.98 \;\; 0.98 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 6.27\cdot kip$ $M_{u.3c.h2d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.1\cdot kip$

$P_{u.3c.h2d2.mn} = 1.79\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 22.7 \;\; -22.7 \;\; 133.2 \;\; 133.2 \;\; 22.7 \;\; -22.7)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (2.92 \;\; 3.1 \;\; 4.42 \;\; 4.42 \;\; 5.23 \;\; 3.1 \;\; 3.91 \;\; 2.75 \;\; 2.45 \;\; 1.65 \;\; 3.26 \;\; 1.8 \;\; 1.5)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.14 \;\; -0.14 \;\; 0.82 \;\; 0.82 \;\; 0.14 \;\; -0.14)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.08 \;\; -0.08 \;\; 0.47 \;\; 0.47 \;\; 0.08 \;\; -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 5.23\cdot kip$ $M_{u.3c.h2d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 0.92\cdot kip$

$P_{u.3c.h2d3.mn} = 1.5\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.14\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 106.5 \;\; 106.5 \;\; 213.1 \;\; 106.5 \;\; 36.3 \;\; -36.3 \;\; 213.1 \;\; 213.1 \;\; 36.3 \;\; -36.3)\cdot plf$

$P_{u.3c.h3d1}^{T} = (4.84 \;\; 5.11 \;\; 7.22 \;\; 7.22 \;\; 8.51 \;\; 5.11 \;\; 6.4 \;\; 4.55 \;\; 4.06 \;\; 2.73 \;\; 5.31 \;\; 2.98 \;\; 2.49)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.65 \;\; 0.65 \;\; 1.31 \;\; 0.65 \;\; 0.22 \;\; -0.22 \;\; 1.31 \;\; 1.31 \;\; 0.22 \;\; -0.22)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.37 \;\; 0.37 \;\; 0.75 \;\; 0.37 \;\; 0.13 \;\; -0.13 \;\; 0.75 \;\; 0.75 \;\; 0.13 \;\; -0.13)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 8.51\cdot kip$ $M_{u.3c.h3d1.mx} = 1.31\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.47\cdot kip$

$P_{u.3c.h3d1.mn} = 2.49\cdot kip$ $M_{u.3c.h3d1.mn} = -0.22\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 79.9 \;\; 79.9 \;\; 159.8 \;\; 79.9 \;\; 27.2 \;\; -27.2 \;\; 159.8 \;\; 159.8 \;\; 27.2 \;\; -27.2)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.63 \;\; 3.83 \;\; 5.42 \;\; 5.42 \;\; 6.38 \;\; 3.83 \;\; 4.8 \;\; 3.41 \;\; 3.04 \;\; 2.05 \;\; 3.98 \;\; 2.23 \;\; 1.87)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.98 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.98 \;\; 0.98 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 6.38\cdot kip$ $M_{u.3c.h3d2.mx} = 0.98\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.1\cdot kip$

$P_{u.3c.h3d2.mn} = 1.87\cdot kip$ $M_{u.3c.h3d2.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 80\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 66.6 \;\; 66.6 \;\; 133.2 \;\; 66.6 \;\; 22.7 \;\; -22.7 \;\; 133.2 \;\; 133.2 \;\; 22.7 \;\; -22.7)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.41 \;\; 0.41 \;\; 0.82 \;\; 0.41 \;\; 0.14 \;\; -0.14 \;\; 0.82 \;\; 0.82 \;\; 0.14 \;\; -0.14)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.23 \;\; 0.23 \;\; 0.47 \;\; 0.23 \;\; 0.08 \;\; -0.08 \;\; 0.47 \;\; 0.47 \;\; 0.08 \;\; -0.08)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 5.32\cdot kip$ $M_{u.3c.h3d3.mx} = 0.82\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 0.92\cdot kip$

$P_{u.3c.h3d3.mn} = 1.55\cdot kip$ $M_{u.3c.h3d3.mn} = -0.14\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - (0.75in + 0.5D_{3c.5})\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.466\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.1\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 0.916\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{(k_{wall}\cdot H_{wall})^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

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$$\delta_{3c.h1d1} := \frac{1}{1 - \frac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.394 \qquad \Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d1} = 0.717$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

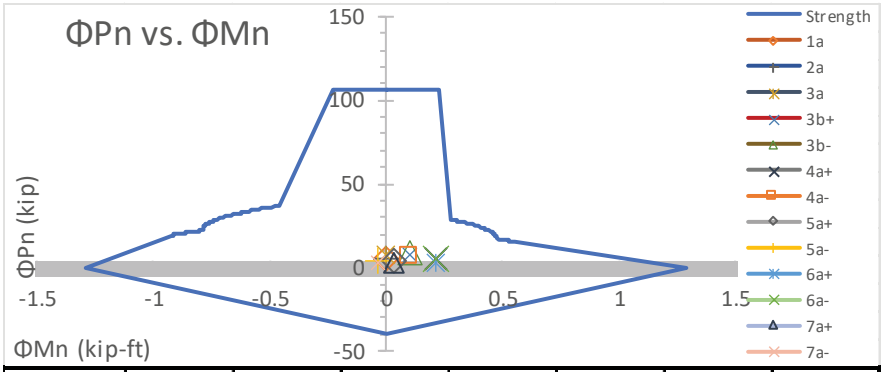
$$\delta_{3c.h2d1} := \frac{1}{1 - \frac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d1} = 1.559 \qquad \Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1} \qquad \Omega_{\delta.3c.h2d1} = 0.642$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h3d1} := \frac{1}{1 - \frac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d1} = 2.649 \qquad \Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1} \qquad \Omega_{\delta.3c.h3d1} = 0.378$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\Omega_{\delta.3c.d1} := \min\left(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1}\right) \qquad \Omega_{\delta.3c.d1} = 0.378$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.5%	4.8%	6.8%	11.41%	12.2%	20.5%	10.9%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.3%	3.8%	18.5%	20.7%	3.2%	2.9%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h1d2} := \frac{1}{1 - \frac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.394 \qquad \Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d2} = 0.717$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h2d2} := \frac{1}{1 - \frac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d2} = 1.368 \qquad \Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1} \qquad \Omega_{\delta.3c.h2d2} = 0.731$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h3d2} := \frac{1}{1 - \frac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d2} = 1.876 \qquad \Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1} \qquad \Omega_{\delta.3c.h3d2} = 0.533$$

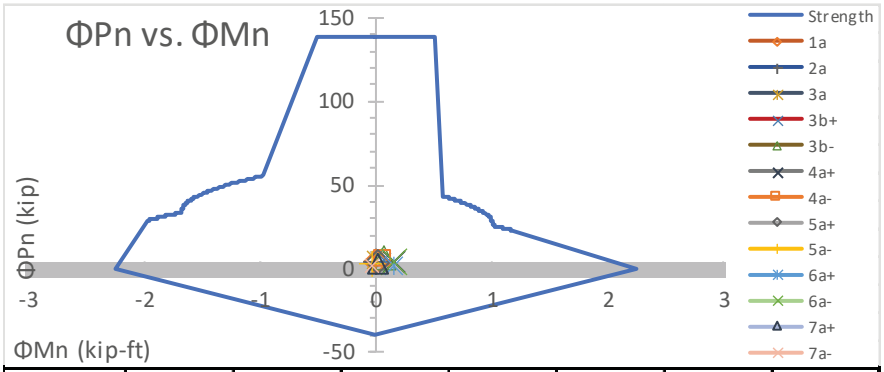
Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\Omega_{\delta.3c.d2} := \min\left(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2}\right) \qquad \Omega_{\delta.3c.d2} = 0.533$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.6%	2.8%	3.9%	4.01%	4.6%	7.7%	4.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.5%	2.2%	7.4%	7.8%	1.6%	1.3%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.215 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.823$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

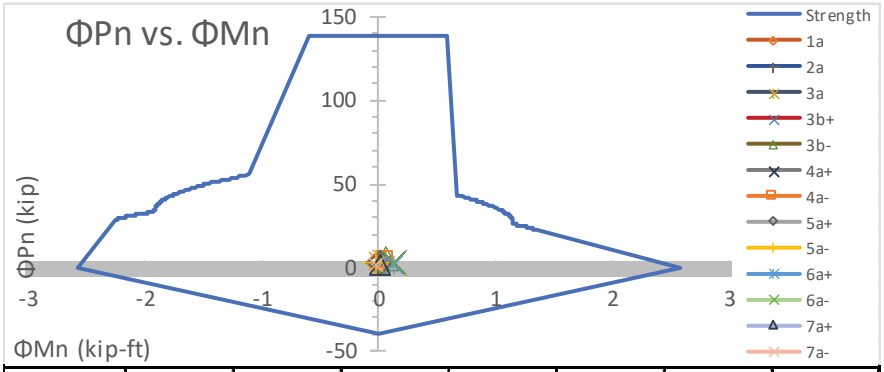
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.289 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.776$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.637 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.611$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.611$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.2%	2.3%	3.3%	3.26%	3.8%	5.5%	2.9%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.1%	1.8%	5.4%	5.6%	1.3%	1.1%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

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d. Wall - Header Check (Above Opening) $w_{llr} = 80 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in$ Min Header Height for Opening Widths of $w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$Wu_{3d.0}^{T} = (745.3 \quad 878.8 \quad 1406.8 \quad 1406.8 \quad 1728.9 \quad 878.8 \quad 1200.9 \quad 738 \quad 615.8 \quad 423.3 \quad 1067.5 \quad 484.4 \quad 362.2) \cdot plf$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1728.9 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in$ $A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for for:
$w_{llr} = 80 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min\left(Vc_{3d.0.a}, Vc_{3d.0.b}\right)$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot \left(w_{open_0}\right)^2}{8}$ $Mu_{3d.0} = 7.78 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max\left(\left|wu_{3d.0mx}\right|, \left|wu_{3d.0mn}\right|\right) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 5.19 \cdot kip$

$chk_{3d.1} := \text{if}\left(\phi Mn_{3d.0} \geq Mu_{3d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot \left[f_y \cdot \left(0.5 \cdot h_{3d_0}\right)\right]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := \text{if}\left(chk_{3d.2} = \text{"OK"}, \text{"OK"}, \text{if}\left(chk_{3d.2} = \text{"Stirrups Req'd"}, \text{if}\left(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, \text{"NG"}, \text{"OK"}\right), \text{"OK"}\right)\right)$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (644.8 \quad 792.7 \quad 1320.7 \quad 1320.7 \quad 1642.8 \quad 792.7 \quad 1114.8 \quad 651.9 \quad 529.7 \quad 367.3 \quad 1011.5 \quad 428.4 \quad 306.2) \cdot plf$

$wu_{3d.1mx} := \max\left(Wu_{3d.1}\right)$ $wu_{3d.1mx} = 1642.8 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min\left(Wu_{3d.1}, 0plf\right)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 80 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}\left(a_{3d.1.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots$$
$$+ A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := \text{root}\left(f_{3d.1}\left(a_{3d.1.g}\right), a_{3d.1.g}\right)$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
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Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F2_{3d.1} := A_{3d.1_1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.1} = 11.7\cdot kip$$

$$F3_{3d.1} := A_{3d.1_2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.1} = 7.2\cdot kip$$

$$F4_{3d.1} := A_{3d.1_3}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1}\cdot\beta_1}\cdot d_{3d.1_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.1} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772\cdot kip$$

$$Mn_{3d.1} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.1}\cdot\frac{a_{3d.1}}{2} + F1_{3d.1}\cdot d_{3d.1_0} + F2_{3d.1}\cdot d_{3d.1_1} \ldots + F3_{3d.1}\cdot d_{3d.1_2} + F4_{3d.1}\cdot d_{3d.1_3} \qquad Mn_{3d.1} = 38.5\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}}\cdot d_{3d.1_3} \qquad \varepsilon_{3d.1} = 0.014 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.1} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.1} := \phi_{3d.1}\cdot Mn_{3d.1} \qquad \phi Mn_{3d.1} = 34.64\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375\cdot in$$

$$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right)\cdot t_w} \qquad \rho_{3d.1} = 0.0062$$

$$\rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{3d.1} := \sum\begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.1_0}\right)}\right] \qquad \rho'_{3d.1} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.1} = 0.876$$

$$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$$

$$Vc_{3d.1.a} := 8\cdot\lambda_{s.3d.1}\cdot\lambda\cdot\rho_{3d.1}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.1_3} \qquad Vc_{3d.1.a} = 5.48\cdot kip$$

$$Vc_{3d.1.b} = 21.31\cdot kip$$

$$\phi Vc_{3d.1} := \phi_V\cdot\min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right) \qquad \phi Vc_{3d.1} = 4.1\cdot kip \qquad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$$

$$Mu_{3d.1} := \frac{wu_{3d.1mx}\cdot\left(w_{open_1}\right)^2}{8} \qquad Mu_{3d.1} = 3.29\cdot kip\cdot ft$$

$$Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right)\cdot w_{open_1}}{2} \qquad Vu_{3d.1} = 3.29\cdot kip$$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h_{3d_1}\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 80 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (623.7 \quad 774.6 \quad 1302.6 \quad 1302.6 \quad 1624.7 \quad 774.6 \quad 1096.7 \quad 633.8 \quad 511.6 \quad 355.5 \quad 999.7 \quad 416.6 \quad 294.4) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1624.7 \cdot plf$ Max. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_2} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_2} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 80 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 3.29 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 2.44 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} :=$ "OK" if $\phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2}$; "Stirrups Req'd" otherwise $\quad chk_{3d.10} = \text{"Min. Stirrups Req'd"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 3.79 \cdot psf$		$M_{E.4a} = 0.001 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.003 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.235 \\ 0.235 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.081 \\ 0.081 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.193 \\ 0.193 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.063 \\ 0.063 \\ 0.006 \\ 0.006 \\ 0.007 \\ 0.007 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.152 \\ 0.152 \\ 0.015 \\ 0.015 \\ 0.018 \\ 0.018 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Bessemer, AL 35022
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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 11.7 \cdot plf$		$M_{E.4b} = 0.127 \cdot kip \cdot ft$	$V_{E.4b} = 0.055 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.53 \\ 6.53 \\ 0.58 \\ 0.58 \\ 0.71 \\ 0.71 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.8 \\ 2.8 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.44 \\ 5.44 \\ 0.58 \\ 0.58 \\ 0.71 \\ 0.71 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.33 \\ 2.33 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.35 \\ 4.35 \\ 0.58 \\ 0.58 \\ 0.71 \\ 0.71 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.86 \\ 1.86 \\ 0.25 \\ 0.25 \\ 0.3 \\ 0.3 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Const. Type: VB
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Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (4 \quad 5 \quad 6 \quad 7 \quad 8 \quad 9 \quad 10 \quad 10 \quad 11)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (11.29 \quad 13.29 \quad 15.29 \quad 17.29 \quad 19.29 \quad 21.29 \quad 23.29 \quad 25.29 \quad 27.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (5.93 \quad 6.3 \quad 6.57 \quad 6.78 \quad 6.95 \quad 7.08 \quad 7.19 \quad 6.62 \quad 6.75) \cdot psf$ Floor Transverse Ribs

$wgt_{f.l.rib}^T = (3.31 \quad 3.38 \quad 3.42 \quad 3.46 \quad 3.49 \quad 3.51 \quad 3.53 \quad 3.61 \quad 3.62) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (8.49 \quad 7.21 \quad 6.27 \quad 5.54 \quad 4.97 \quad 4.5 \quad 4.12 \quad 3.79 \quad 3.51) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (3.74 \quad 3.84 \quad 3.91 \quad 3.96 \quad 4.01 \quad 4.04 \quad 4.07 \quad 4.09 \quad 4.11) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (41.1 \quad 40.4 \quad 39.8 \quad 39.4 \quad 39.1 \quad 38.8 \quad 38.6 \quad 37.8 \quad 37.7) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (45.1 \quad 44.4 \quad 43.8 \quad 43.4 \quad 43.1 \quad 42.8 \quad 42.6 \quad 41.8 \quad 41.7) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

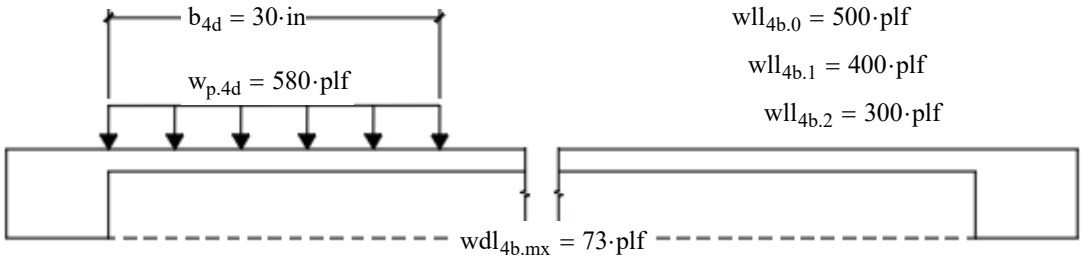
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

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$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
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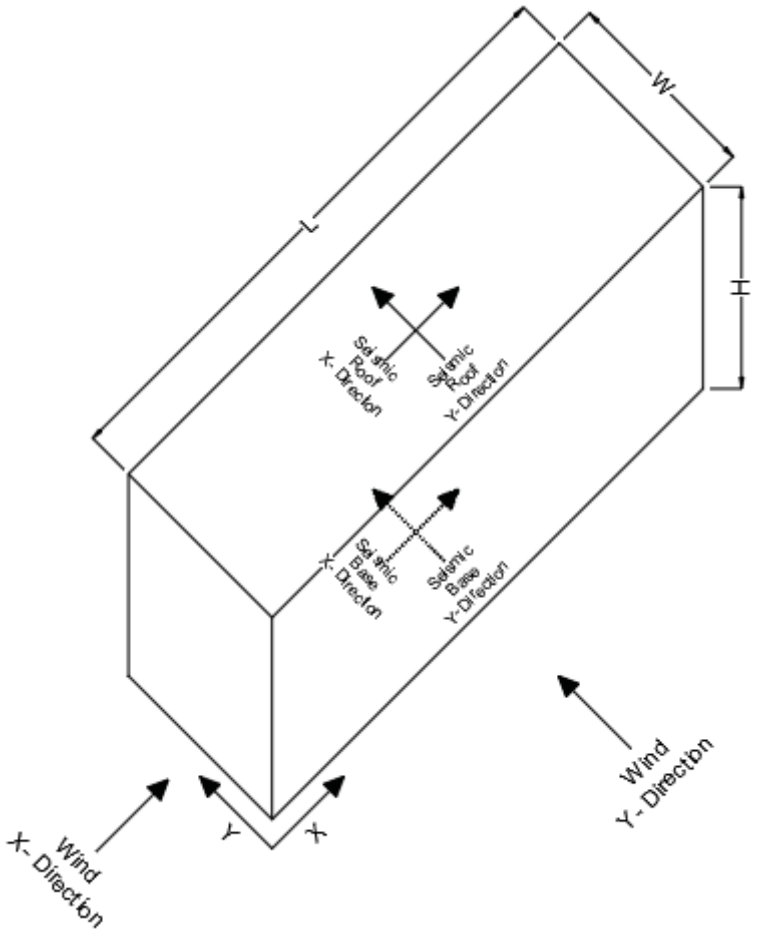
5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx} \cdot (L_r) \cdot (W_r)$

$Wt_{rf.mn} := DL_{r.mn} \cdot (L_r) \cdot (W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25 \cdot w_{llr}, 0psf)$

$w_{llr.eq} = 20 \cdot psf$

$Wt_{25ll} := w_{llr.eq} \cdot [(L_r) \cdot (W_r)]$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{rf.mx}^{T} = (9.4 \quad 10.9 \quad 12.5 \quad 14 \quad 15.5 \quad 17 \quad 18.6 \quad 20.1 \quad 21.6) \cdot kip$

Max Roof Weight, $w_{llr} = 80 \cdot psf$

$Wt_{25ll}^{T} = (3 \quad 3.4 \quad 3.9 \quad 4.4 \quad 4.9 \quad 5.4 \quad 5.8 \quad 6.3 \quad 6.8) \cdot kip$

25% Roof LLr 0, $w_{llr} = 80 \cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28) \cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (8.4 \quad 9.7 \quad 11.1 \quad 12.4 \quad 13.8 \quad 15.2 \quad 16.5 \quad 17.9 \quad 19.2) \cdot kip$

Min Roof Weight, $w_{llr} = 80 \cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75 \cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3} \cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128) \cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (114.5 \quad 126.5 \quad 134.5) \cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 111.25 \\ 123.25 \\ 131.25 \end{pmatrix} \cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r} \cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r} \cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r} \cdot el_{avg.rf_2}$

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Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{w.h0.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{EQ.w.h0}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{EQ.w.h1}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{EQ.w.h2}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

$el_{top.w} := h_{wl\_fl} + H_{wall}$ $el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$ Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean\left(h_{wl\_fl}, el_{top.w_0}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_1}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_2}\right) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$ $W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$ $W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if\left(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf\right)$ $w_{llf.0.eq} = 62.5 \cdot psf$ $Wt_{25llf0} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.1.eq} := if\left(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf\right)$ $w_{llf.1.eq} = 50 \cdot psf$ $Wt_{25llf1} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.2.eq} := if\left(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf\right)$ $w_{llf.2.eq} = 37.5 \cdot psf$ $Wt_{25llf2} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$ $Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (5.6 \quad 6.5 \quad 7.3 \quad 8.2 \quad 9.1 \quad 10 \quad 10.9 \quad 11.6 \quad 12.5) \cdot kip$ Max Floor Weight

$Wt_{f.mn}^T = (5.1 \quad 5.9 \quad 6.7 \quad 7.5 \quad 8.3 \quad 9.1 \quad 9.8 \quad 10.5 \quad 11.3) \cdot kip$ Min Floor Weight

$Wt_{25llf0}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$ $Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$ $Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$ Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$ Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$Wt_{EQ.f.f0}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 45.99\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (5 \quad 5.8 \quad 6.7 \quad 7.6 \quad 8.5 \quad 9.4 \quad 10.2 \quad 11.1 \quad 12)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (5.5 \quad 6.5 \quad 7.4 \quad 8.4 \quad 9.4 \quad 10.3 \quad 11.3 \quad 12.3 \quad 13.3)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (5.8 \quad 6.9 \quad 7.9 \quad 8.9 \quad 10 \quad 11 \quad 12 \quad 13.1 \quad 14.1)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 5.13\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 5.67\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 6.02\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (2.5 \quad 2.9 \quad 3.4 \quad 3.8 \quad 4.2 \quad 4.7 \quad 5.1 \quad 5.6 \quad 6)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (2.7 \quad 3.2 \quad 3.7 \quad 4.2 \quad 4.7 \quad 5.2 \quad 5.7 \quad 6.1 \quad 6.6)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (2.9 \quad 3.4 \quad 3.9 \quad 4.5 \quad 5 \quad 5.5 \quad 6 \quad 6.5 \quad 7)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 2.57\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 2.83\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 3.01\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.2$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (44 \quad 50 \quad 56 \quad 62 \quad 68 \quad 74 \quad 80 \quad 85 \quad 91)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$V_{EQ.bh0f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh1f0} := C_s\cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f0}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh2f0} := C_s\cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh0f1}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh0f1} := C_s\cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f1}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh1f1} := C_s\cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f1}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh2f1} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh0f2}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh0f2} := C_s\cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh1f2}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh1f2} := C_s\cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.bh2f2}^{T} = (44 \;\; 50 \;\; 56 \;\; 62 \;\; 68 \;\; 74 \;\; 80 \;\; 85 \;\; 91)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh2f2} := C_s\cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (9 \;\; 10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 17 \;\; 18)\cdot kip$

$Wt_{EQ.rh0}^{T} = (22 \;\; 25 \;\; 27 \;\; 30 \;\; 33 \;\; 36 \;\; 39 \;\; 41 \;\; 44)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 80\cdot psf$ $H_{wall_0} = 9\,ft$

$V_{EQ.rh0} := C_s\cdot Wt_{EQ.rh0}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$V_{EQ.rh0}^T = (4 \quad 5 \quad 5 \quad 6 \quad 7 \quad 7 \quad 8 \quad 8 \quad 9)\cdot kip$

$Wt_{EQ.rh1}^T = (22 \quad 25 \quad 27 \quad 30 \quad 33 \quad 36 \quad 39 \quad 41 \quad 44)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^T = (4 \quad 5 \quad 5 \quad 6 \quad 7 \quad 7 \quad 8 \quad 8 \quad 9)\cdot kip$

$Wt_{EQ.rh2}^T = (22 \quad 25 \quad 27 \quad 30 \quad 33 \quad 36 \quad 39 \quad 41 \quad 44)\cdot kip$

Roof Diap. Wt/Shear $w_{llr} = 80\cdot psf$

$H_{wall_2} = 10.7\,ft$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^T = (4 \quad 5 \quad 5 \quad 6 \quad 7 \quad 7 \quad 8 \quad 8 \quad 9)\cdot kip$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r}$ $w_{EQh0.y}^T = (353 \quad 343 \quad 336 \quad 330 \quad 325 \quad 321 \quad 318 \quad 315 \quad 313)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r}$ $w_{EQh1.y}^T = (353 \quad 343 \quad 336 \quad 330 \quad 325 \quad 321 \quad 318 \quad 315 \quad 313)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r}$ $w_{EQh2.y}^T = (353 \quad 343 \quad 336 \quad 330 \quad 325 \quad 321 \quad 318 \quad 315 \quad 313)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r}$ $w_{EQh0.x}^T = (363 \quad 410 \quad 457 \quad 504 \quad 551 \quad 598 \quad 645 \quad 692 \quad 739)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r}$ $w_{EQh1.x}^T = (363 \quad 410 \quad 457 \quad 504 \quad 551 \quad 598 \quad 645 \quad 692 \quad 739)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r}$ $w_{EQh2.x}^T = (363 \quad 410 \quad 457 \quad 504 \quad 551 \quad 598 \quad 645 \quad 692 \quad 739)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 80\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r}$ $w_{W.h0.y}^T = (201 \quad 204 \quad 206 \quad 207 \quad 209 \quad 210 \quad 210 \quad 211 \quad 212)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r}$ $w_{W.h1.y}^T = (222 \quad 225 \quad 227 \quad 229 \quad 230 \quad 232 \quad 232 \quad 233 \quad 234)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r}$ $w_{W.h2.y}^T = (236 \quad 239 \quad 242 \quad 244 \quad 245 \quad 246 \quad 247 \quad 248 \quad 249)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67\,ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$ $w_{W.h0.x} = 213.8\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$ $w_{W.h1.x} = 236.1\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$ $w_{W.h2.x} = 251\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_2} = 10.7\,ft$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$M_{u.bh0.y}^{T} = (6.7 \;\; 8.8 \;\; 11.2 \;\; 13.9 \;\; 16.8 \;\; 20 \;\; 23.6 \;\; 27.3 \;\; 31.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (6.7 \;\; 8.8 \;\; 11.2 \;\; 13.9 \;\; 16.8 \;\; 20 \;\; 23.6 \;\; 27.3 \;\; 31.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (6.7 \;\; 8.8 \;\; 11.2 \;\; 13.9 \;\; 16.8 \;\; 20 \;\; 23.6 \;\; 27.3 \;\; 31.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.y}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.bh0.x}^{T} = (6.5 \;\; 7.4 \;\; 8.2 \;\; 9.1 \;\; 9.9 \;\; 10.8 \;\; 11.6 \;\; 12.5 \;\; 13.3)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (6.5 \;\; 7.4 \;\; 8.2 \;\; 9.1 \;\; 9.9 \;\; 10.8 \;\; 11.6 \;\; 12.5 \;\; 13.3)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (6.5 \;\; 7.4 \;\; 8.2 \;\; 9.1 \;\; 9.9 \;\; 10.8 \;\; 11.6 \;\; 12.5 \;\; 13.3)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7\,ft$

$V_{u.bh0.x}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 80\cdot psf$, $H_{wall_2} = 10.7\,ft$

$M_{u.b.h0.y}^{T} = (3.8 \;\; 5.2 \;\; 6.9 \;\; 8.7 \;\; 10.8 \;\; 13.1 \;\; 15.6 \;\; 18.3 \;\; 21.3)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (4.2 \;\; 5.8 \;\; 7.6 \;\; 9.6 \;\; 11.9 \;\; 14.4 \;\; 17.2 \;\; 20.2 \;\; 23.5)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (4.5 \;\; 6.1 \;\; 8.1 \;\; 10.2 \;\; 12.7 \;\; 15.3 \;\; 18.3 \;\; 21.5 \;\; 25)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7\,ft$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
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$V_{u.b.h0.y}^{T} = (1.2 \quad 1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.6 \quad 2.8 \quad 3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (1.4 \quad 1.6 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.6 \quad 2.8 \quad 3.1 \quad 3.3)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (1.5 \quad 1.7 \quad 2 \quad 2.2 \quad 2.5 \quad 2.7 \quad 3 \quad 3.3 \quad 3.5)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 3.8\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 4.3\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 4.5\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

$v_{u.b.h0.x} = 1.3\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.4\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7\,ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = "\#6"$ $B_{7.1.ref} = "Roof\ Perim.\ Reinf."$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = "\#\ Roof\ Perim.\ Bars"$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = "Roof\ Perim.\ Reiinf.\ Bar\ Diam."$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = "Roof\ Perim.\ Reinf.\ Area"$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := "Roof\ Diaph.Reinf."$ $\rho_{b.1} := \frac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00089$

$d_{b.1} := W_r - 0.75in - \frac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \frac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 2.01\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \frac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\frac{d_{b.1}}{2} - \frac{a_{b.1}}{2}\right)$ $M_{b.1} = 382.5\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \frac{a_{b.1}}{0.85}$ $c_{b.1} = 2.4\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\frac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.17845$ Strain in Ten. Rebar

$\phi_{b.1} := if\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, if\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 344.3\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\frac{M_{u.bh0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.bh1.y}}{\phi M_{b.1_1}} \quad \frac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 1.9 & 2.6 & 3.3 & 4 & 4.9 & 5.8 & 6.8 & 7.9 & 9.1 \\ 1.9 & 2.6 & 3.3 & 4 & 4.9 & 5.8 & 6.8 & 7.9 & 9.1 \\ 1.9 & 2.6 & 3.3 & 4 & 4.9 & 5.8 & 6.8 & 7.9 & 9.1 \end{pmatrix}\cdot\%$

$chk_{b.1a} := if\left(\max\left(Unity_{b.1a}\right) < 1, "OK", "NG"\right)$

$chk_{b.1a} = "OK"$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.1b}^T = \begin{pmatrix} 1.1 & 1.5 & 2 & 2.5 & 3.1 & 3.8 & 4.5 & 5.3 & 6.2 \\ 1.2 & 1.7 & 2.2 & 2.8 & 3.5 & 4.2 & 5 & 5.9 & 6.8 \\ 1.3 & 1.8 & 2.3 & 3 & 3.7 & 4.5 & 5.3 & 6.2 & 7.2 \end{pmatrix} \cdot \%$$

$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.64 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof\ Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 2.008 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$

$d_{b.2}^T = (12.26 \quad 14.26 \quad 16.26 \quad 18.26 \quad 20.26 \quad 22.26 \quad 24.26 \quad 26.26 \quad 28.26)\,ft$

$\rho_{b.2}^T = (0.00086 \quad 0.00074 \quad 0.00065 \quad 0.00058 \quad 0.00052 \quad 0.00048 \quad 0.00044 \quad 0.00041 \quad 0.00038)$

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.2}}{2} \right)$$

$M_{b.2a}^T = (394 \quad 459 \quad 524 \quad 588 \quad 653 \quad 718 \quad 783 \quad 848 \quad 912) \cdot kip \cdot ft$ Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 2.4 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$\varepsilon_{t.b.2a}^T = (0.1839 \quad 0.2143 \quad 0.2448 \quad 0.2753 \quad 0.3058 \quad 0.3363 \quad 0.3667 \quad 0.3972 \quad 0.4277)$ Strain in Rebar

$\phi_{b.2} := if\left[min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50 \right] \right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^T = (355 \quad 413 \quad 471 \quad 530 \quad 588 \quad 646 \quad 705 \quad 763 \quad 821) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.2a}^T = \begin{pmatrix} 1.8 & 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.8 & 1.7 & 1.7 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.2b}^T = \begin{pmatrix} 1.1 & 0.9 & 0.8 & 0.7 & 0.7 & 0.6 & 0.5 & 0.5 & 0.5 \\ 1.2 & 1 & 0.9 & 0.8 & 0.7 & 0.7 & 0.6 & 0.6 & 0.5 \\ 1.3 & 1.1 & 1 & 0.9 & 0.8 & 0.7 & 0.6 & 0.6 & 0.6 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 720 \cdot in^2$ Roof Shear Area for $t_{r.e} = 5 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00089$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e}\cdot L_{r_0} \\ t_{r.e}\cdot L_{r_1} \\ t_{r.e}\cdot L_{r_2} \\ t_{r.e}\cdot L_{r_3} \\ t_{r.e}\cdot L_{r_4} \\ t_{r.e}\cdot L_{r_5} \\ t_{r.e}\cdot L_{r_6} \\ t_{r.e}\cdot L_{r_7} \\ t_{r.e}\cdot L_{r_8} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 740 \\ 860 \\ 980 \\ 1100 \\ 1220 \\ 1340 \\ 1460 \\ 1580 \\ 1700 \end{pmatrix}\cdot in^2$$

Roof Shear Area for $t_{r.e} = 5\cdot in$ & $L_r = \begin{pmatrix} 1776 \\ 2064 \\ 2352 \\ 2640 \\ 2928 \\ 3216 \\ 3504 \\ 3792 \\ 4080 \end{pmatrix}\cdot\frac{in^2}{ft}$

$\rho_{b.2}^{T} = (0.00086 \quad 0.00074 \quad 0.00065 \quad 0.00058 \quad 0.00052 \quad 0.00048 \quad 0.00044 \quad 0.00041 \quad 0.00038)$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv}\cdot\left(2\cdot\sqrt{f'_c} + \rho\cdot f_y\right)$ $V_{b.3a.1.mx} = 135\cdot kip$

Max Allowable for $t_{r.e} = 5\cdot in$ & $W_r = 12\cdot ft$

$V_{b.3a.2.mx}^{T} = (137.7 \quad 153.8 \quad 169.9 \quad 186 \quad 202.1 \quad 218.2 \quad 234.3 \quad 250.4 \quad 266.5)\cdot kip$

Max Allowable Shear for $t_{r.e} = 5\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$V_{b.3b.1} = 41.1\cdot kip$ Nominal Shear Strength for $t_{r.e} = 5\cdot in$ & $W_r = 12\cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

Roof Diaph. Shear Strength for $t_{r.e} = 5\cdot in$ & $W_r = 12\cdot ft$ $\phi V_{c.b.3a.1} = 30.8\cdot kip$

$V_{b.3b.2}^{T} = (42.2 \quad 49 \quad 55.9 \quad 62.7 \quad 69.6 \quad 76.4 \quad 83.2 \quad 90.1 \quad 96.9)\cdot kip$

Nominal Shear Strength for $t_{r.e} = 5\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$\phi V_{c.b.3a.2}^{T} = (31.6 \quad 36.8 \quad 41.9 \quad 47 \quad 52.2 \quad 57.3 \quad 62.4 \quad 67.6 \quad 72.7)\cdot kip$

Roof Diaph. Shear Strength for $t_{r.e} = 5\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80\cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 7.1 & 8 & 8.9 & 9.8 & 10.7 & 11.7 & 12.6 & 13.5 & 14.4 \\ 7.1 & 8 & 8.9 & 9.8 & 10.7 & 11.7 & 12.6 & 13.5 & 14.4 \\ 7.1 & 8 & 8.9 & 9.8 & 10.7 & 11.7 & 12.6 & 13.5 & 14.4 \end{pmatrix}\cdot\%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 80\cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 4 & 4.7 & 5.5 & 6.2 & 6.9 & 7.6 & 8.3 & 9 & 9.7 \\ 4.5 & 5.2 & 6 & 6.8 & 7.6 & 8.4 & 9.2 & 10 & 10.8 \\ 4.7 & 5.6 & 6.4 & 7.2 & 8.1 & 8.9 & 9.8 & 10.6 & 11.4 \end{pmatrix}\cdot\%$$

$chk_{b.3d} = "OK"$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \;\; 10 \;\; 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

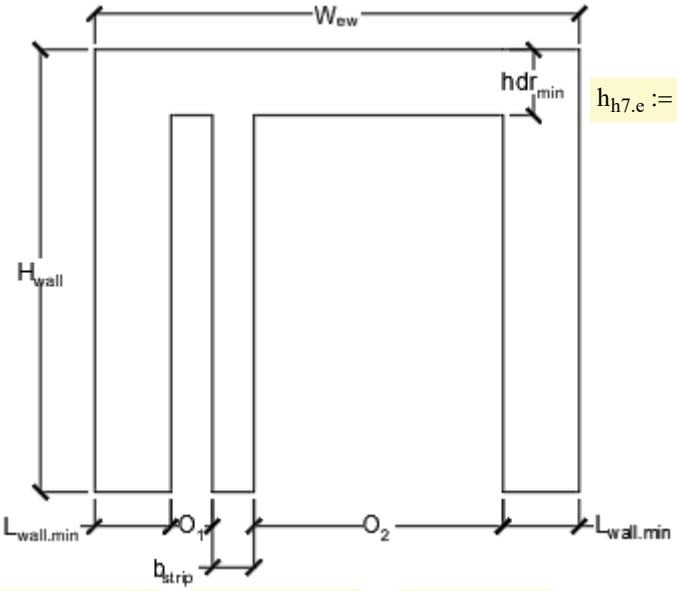
$Unity_{b.3g}^{T} = (6.9 \;\; 6.7 \;\; 6.5 \;\; 6.4 \;\; 6.3 \;\; 6.3 \;\; 6.2 \;\; 6.1 \;\; 6.1) \cdot \%$ $chk_{b.3j} = "OK"$

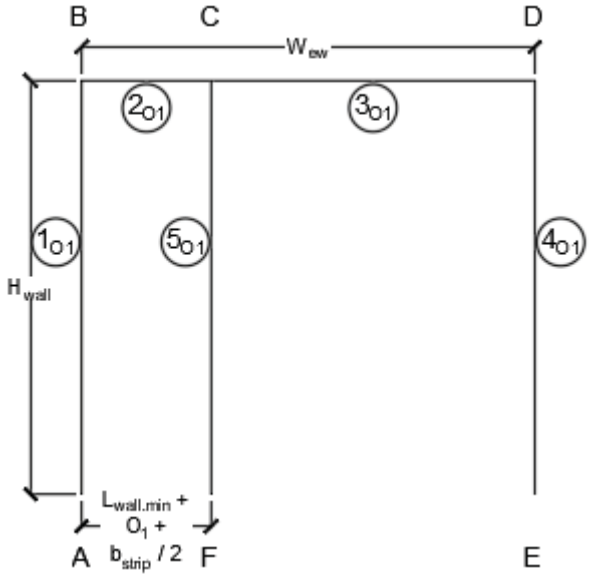
Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \;\; 10 \;\; 10.7)\,ft$ & $w_{llr} = 80 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 4.1 & 3.5 & 3.1 & 2.7 & 2.5 & 2.2 & 2.1 & 1.9 & 1.8 \\ 4.5 & 3.9 & 3.4 & 3 & 2.7 & 2.5 & 2.3 & 2.1 & 1.9 \\ 4.8 & 4.1 & 3.6 & 3.2 & 2.9 & 2.6 & 2.4 & 2.2 & 2.1 \end{pmatrix} \cdot \%$$

$chk_{b.3j} = "OK"$

END WALL DESIGN





$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} = "Modulus of Elas. Conc."$

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right)$$ $Veq_{ewho}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip$ Height $H_{wall_0} = 9\,ft$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right)$$ $Veq_{ewh1}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip$ Height $H_{wall_1} = 10\,ft$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right)$$ $Veq_{ewh2}^{T} = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4) \cdot kip$ Height $H_{wall_2} = 10.7\,ft$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
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Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \frac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \frac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint C

$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix}$ $K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \frac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \frac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}{}^T = (3.7 \;\; 4.1 \;\; 4.6 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.5 \;\; 7 \;\; 7.5) \cdot kip \cdot ft$

$V_{L.ewh0}{}^T = (0.7 \;\; 0.8 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.4) \cdot kip$

$M_{hdrh0}{}^T = (2.7 \;\; 3 \;\; 3.4 \;\; 3.7 \;\; 4.1 \;\; 4.4 \;\; 4.8 \;\; 5.1 \;\; 5.5) \cdot kip \cdot ft$

$V_{hdrh0}{}^T = (0.9 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.5 \;\; 1.7 \;\; 1.8 \;\; 1.9) \cdot kip$

$M_{b.striph0}{}^T = (4 \;\; 4.6 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.7 \;\; 7.2 \;\; 7.7 \;\; 8.2) \cdot kip \cdot ft$

$V_{b.striph0}{}^T = (0.8 \;\; 0.9 \;\; 1 \;\; 0.9 \;\; 1 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10\,ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$ $DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}{}^T = (4.2 \quad 4.8 \quad 5.3 \quad 5.9 \quad 6.4 \quad 7 \quad 7.5 \quad 8.1 \quad 8.6) \cdot kip \cdot ft$

$M_{hdrh1}{}^T = (3.4 \quad 3.8 \quad 4.2 \quad 4.7 \quad 5.1 \quad 5.5 \quad 6 \quad 6.4 \quad 6.9) \cdot kip \cdot ft$

$M_{b.striph1}{}^T = (4.5 \quad 5.1 \quad 5.7 \quad 6.3 \quad 6.9 \quad 7.5 \quad 8 \quad 8.6 \quad 9.2) \cdot kip \cdot ft$

$V_{L.ewh1}{}^T = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.3 \quad 1.4 \quad 1.5) \cdot kip$

$V_{hdrh1}{}^T = (1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.5 \quad 2.7 \quad 2.9 \quad 3.1) \cdot kip$

$V_{b.striph1}{}^T = (0.8 \quad 1 \quad 1.1 \quad 0.8 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6 \quad 1.7) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
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Wind Velocity:	150 MPH (Vult) [165 MPH (V
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Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119 \cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

$M_{L.ewh2}^T = (4.5\ \ 5.1\ \ 5.7\ \ 6.3\ \ 6.9\ \ 7.5\ \ 8.1\ \ 8.6\ \ 9.2)\cdot kip\cdot ft$ $V_{L.ewh2}^T = (0.8\ \ 0.9\ \ 1\ \ 1.1\ \ 1.2\ \ 1.3\ \ 1.4\ \ 1.5\ \ 1.6)\cdot kip$

$M_{hdrh2}^T = (3.7\ \ 4.1\ \ 4.6\ \ 5.1\ \ 5.6\ \ 6\ \ 6.5\ \ 7\ \ 7.4)\cdot kip\cdot ft$ $V_{hdrh2}^T = (1.6\ \ 1.8\ \ 2.1\ \ 2.3\ \ 2.5\ \ 2.7\ \ 2.9\ \ 3.1\ \ 3.3)\cdot kip$

$M_{b.striph2}^T = (4.8\ \ 5.5\ \ 6.1\ \ 6.7\ \ 7.3\ \ 8\ \ 8.6\ \ 9.2\ \ 9.8)\cdot kip\cdot ft$ $V_{b.striph2}^T = (0.9\ \ 1\ \ 1.1\ \ 0.8\ \ 1.3\ \ 1.4\ \ 1.5\ \ 1.6\ \ 1.7)\cdot kip$

Reinforcement

$B_{3.1} = \text{"\#6"}$ $B_{3.1.ref} = \text{"Wall Vert. Perim. Reinf."}$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = \text{"\# Wall Vert. Perim. Bars"}$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.3} = \text{"\#6"}$ $B_{3.3.ref} = \text{"EW Bot Perim. Reinf."}$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = \text{"\# EW Bot Perim. Bars"}$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.4} = \text{"\#4"}$ $B_{3.4.ref} = \text{"SW Bot Perim. Reinf."}$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = \text{"\# SW Bot Perim. Bars"}$

$D_{3.4} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17\cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5\cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5\cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{ewc} = 1.7\cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{h.e} = 1.6\cdot in$

$a_{strip} := \frac{A_{3.5}\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{strip} = 2.4\cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot L_{ew.mn}$ $A_{mn.e} = 0.079\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.h.e} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.strip} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w\cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w\cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w\cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

$chk_{c.1} := if\left(A_{mn.e} \le \min\left(A_{3.1}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.1} = "OK"$

$chk_{c.2} := if\left(A_{mn.e} \le \min\left(A_{3.2}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.2} = "OK"$

$chk_{c.3} := if\left(A_{mn.strip} \le A_{3.5}, "OK", "NG"\right)$ $chk_{c.3} = "OK"$

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = "Tensile Strain, Extreme Layer"$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{ewc} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{h.e} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{strip} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi Mn_{ewc} := \phi_{ewc} \cdot \min\left(A_{3.1}, A_{3.5}\right) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min\left(A_{3.2}, A_{3.5}\right) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref} = "Flexural Strength, +ve, Sect (22.3)"$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"$

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$Vc_{ewc.1} := 8\cdot\lambda\cdot\lambda_{s.ewc}\cdot\rho_{ewc}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975\cdot kip$ $Vc_{ewc.2} = 19.387\cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V\cdot\min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2\cdot\left(t_w\cdot d_{ewc}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{ewc} = 3.7\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8\cdot\lambda\cdot\lambda_{s.h.e}\cdot\rho_{h.e}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663\cdot kip$ $Vc_{h.e.2} = 17.676\cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V\cdot\min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2\cdot\left(t_w\cdot d_{h7.e}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{h.e} = 3.5\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8\cdot\lambda\cdot\lambda_{s.strip}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{strip}\right)$ $Vc_{strip.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347\cdot kip$ $Vc_{strip.2} = 26.799\cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V\cdot\min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2\cdot\left(t_w\cdot d_{h7.e}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.19 & 0.2 & 0.21 & 0.11 & 0.13 & 0.14 \\ 0.21 & 0.23 & 0.23 & 0.13 & 0.15 & 0.16 \\ 0.24 & 0.26 & 0.26 & 0.14 & 0.17 & 0.18 \\ 0.26 & 0.28 & 0.29 & 0.16 & 0.18 & 0.2 \\ 0.29 & 0.31 & 0.31 & 0.17 & 0.2 & 0.22 \\ 0.31 & 0.34 & 0.34 & 0.19 & 0.22 & 0.23 \\ 0.34 & 0.36 & 0.37 & 0.2 & 0.24 & 0.25 \\ 0.36 & 0.39 & 0.39 & 0.22 & 0.25 & 0.27 \\ 0.39 & 0.41 & 0.42 & 0.23 & 0.27 & 0.29 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

$$Uny_{strip} = \begin{pmatrix} 0.1 & 0.1 & 0.1 & 0.07 & 0.07 & 0.08 \\ 0.11 & 0.12 & 0.12 & 0.07 & 0.08 & 0.09 \\ 0.13 & 0.13 & 0.13 & 0.08 & 0.09 & 0.1 \\ 0.11 & 0.09 & 0.09 & 0.09 & 0.1 & 0.11 \\ 0.12 & 0.16 & 0.16 & 0.1 & 0.11 & 0.12 \\ 0.13 & 0.17 & 0.17 & 0.11 & 0.12 & 0.13 \\ 0.14 & 0.18 & 0.18 & 0.12 & 0.13 & 0.14 \\ 0.15 & 0.19 & 0.2 & 0.12 & 0.14 & 0.15 \\ 0.16 & 0.21 & 0.21 & 0.13 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$Uny_{hdr} = \begin{pmatrix} 0.27 & 0.43 & 0.47 & 0.1 & 0.13 & 0.14 \\ 0.3 & 0.48 & 0.53 & 0.11 & 0.14 & 0.16 \\ 0.34 & 0.54 & 0.59 & 0.13 & 0.16 & 0.17 \\ 0.37 & 0.6 & 0.65 & 0.14 & 0.18 & 0.19 \\ 0.41 & 0.65 & 0.71 & 0.15 & 0.19 & 0.21 \\ 0.44 & 0.71 & 0.77 & 0.17 & 0.21 & 0.23 \\ 0.48 & 0.76 & 0.83 & 0.18 & 0.23 & 0.25 \\ 0.51 & 0.82 & 0.89 & 0.19 & 0.24 & 0.26 \\ 0.55 & 0.87 & 0.95 & 0.21 & 0.26 & 0.28 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

$Vuh1^T = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

$Vuh2^T = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Wall Plates: $No_{plates} := 1 \quad V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$

$V_{u.Floor.PLh0}{}^T = (3.4 \;\; 3.8 \;\; 4.2 \;\; 4.7 \;\; 5.1 \;\; 5.5 \;\; 6 \;\; 6.4 \;\; 6.8)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$

$V_{u.Floor.PLh1}{}^T = (3.7 \;\; 4.2 \;\; 4.7 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.6 \;\; 7.1 \;\; 7.6)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$

$V_{u.Floor.PLh2}{}^T = (4 \;\; 4.5 \;\; 5 \;\; 5.5 \;\; 6 \;\; 6.6 \;\; 7.1 \;\; 7.6 \;\; 8.1)\cdot kip$

$A_{cv.b.3b.1}{}^T = (740 \;\; 860 \;\; 980 \;\; 1100 \;\; 1220 \;\; 1340 \;\; 1460 \;\; 1580 \;\; 1700)\cdot in^2$

$\rho_{b.2}{}^T = (0.00086 \;\; 0.00074 \;\; 0.00065 \;\; 0.00058 \;\; 0.00052 \;\; 0.00048 \;\; 0.00044 \;\; 0.00041 \;\; 0.00038)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \quad \Longrightarrow \quad 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force $v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.1.2h0}{}^T = (2.2 \;\; 2.5 \;\; 2.7 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.4)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.2.2h1}{}^T = (2.4 \;\; 2.7 \;\; 3 \;\; 3.4 \;\; 3.7 \;\; 4 \;\; 4.3 \;\; 4.6 \;\; 4.9)\cdot\dfrac{kip}{ft}$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.co

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$v_{u.5d.3.2h2} := \frac{V_{u.Wall.PLh2}}{\left\lceil \left(L_{ew.mn}\right) - 3.5in \right\rceil}$$

$$v_{u.5d.3.2h2}^{T} = (2.6 \quad 2.9 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.3 \quad 4.6 \quad 4.9 \quad 5.3)\cdot\frac{kip}{ft}$$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.2.2h1}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.3.2h2}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}\right)^{T} \qquad Hwall := \left(H_{wall_0} \quad H_{wall_1} \quad H_{wall_2}\right)$$

$$Unity_{5d1.2.3}^{T} = \begin{pmatrix} 33.82 & 38.21 & 42.6 & 46.99 & 51.38 & 55.77 & 60.16 & 64.55 & 68.94 \\ 37.58 & 42.46 & 47.33 & 52.21 & 57.09 & 61.97 & 66.85 & 71.73 & 76.6 \\ 40.08 & 45.29 & 50.49 & 55.69 & 60.9 & 66.1 & 71.3 & 76.51 & 81.71 \end{pmatrix}\cdot\% \qquad Hwall^{T} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$$

$$L^{T} = (12.00 \quad 14.00 \quad 16.00 \quad 18.00 \quad 20.00 \quad 22.00 \quad 24.00 \quad 26.00 \quad 28.00)\,ft$$

$chk_{5d.1} := if\left(1.0 \geq max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4\cdot in$ Wall Thickness $w_{plate} = 16\cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \end{pmatrix} \qquad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix}\cdot in$$

$b_{eff.5d.4} := b_{eff.5d_0}$ $b_{eff.5d.5} := b_{eff.5d_1}$ $b_{eff.5d.6} := b_{eff.5d_2}$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$	$b_{5d.4} = 8.3\,ft$	For Wall Panel Height of
$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.4} := H_{wall_0} - t_{floor}$	$l_{c.5d.4} = 8.3\,ft$	$H_{wall_0} = 9\,ft$
$a_{5d5} := W_{ew}$	$a_{5d5} = 11.7\,ft$	$b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$	$b_{5d.5} = 9.3\,ft$	For Wall Panel Height of
$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.5} := H_{wall_1} - t_{floor}$	$l_{c.5d.5} = 9.3\,ft$	$H_{wall_1} = 10\,ft$
$a_{5d6} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$	$b_{5d.6} = 10\,ft$	For Wall Panel Height of
$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$	$a\_b_{5d.6} = 1.2$	$l_{c.5d.6} := H_{wall_2} - t_{floor}$	$l_{c.5d.6} = 10\,ft$	$H_{wall_2} = 10.7ft$

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \qquad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft \qquad P_{5.5} := max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right) \qquad P_{5.5} = 53.3\cdot psf$$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4}\cdot P_{5.5}\cdot\left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 15.08\cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6}\cdot 1\,ft$ $S_{5d.4} = 32\cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4}\cdot S_{5d.4}$ $Mu_{5d.4} = 3.4\cdot kip\cdot in$

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Assume only (1) end wall for resistance to lateral load

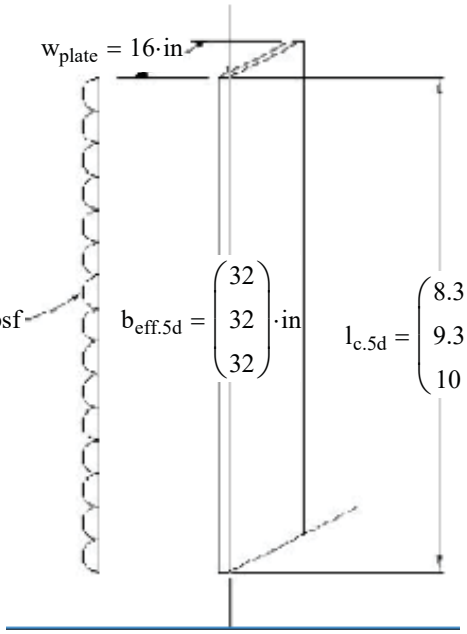
$P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}^{T} = (6.7 \;\; 7.6 \;\; 8.5 \;\; 9.3 \;\; 10.2 \;\; 11.1 \;\; 11.9)$ $P_{5d.4max} := \max(2 \cdot V_{u.Wall.PLh0})$ $P_{5d.4max} = 13.7 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}^{T} = (3.4 \;\; 3.8 \;\; 4.2 \;\; 4.7 \;\; 5.1 \;\; 5.5 \;\; 6 \;\; 6.4 \;\; 6.8) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \dfrac{\beta_{5d.5} \cdot P_{5.5} \cdot (l_{c.5d.5})^2}{(t_w)^2} = 18.92 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \dfrac{\beta_{5d.6} \cdot P_{5.5} \cdot (l_{c.5d.6})^2}{(t_w)^2} = 17.33 \cdot ksf$



$S_{5d.5} := \dfrac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 4.2 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15.2 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \dfrac{(t_w)^2}{6} \cdot 1ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 3.9 \cdot kip \cdot in$

$P_{5d.5}^{T} = (7.5 \;\; 8.4 \;\; 9.4 \;\; 10.4 \;\; 11.3 \;\; 12.3 \;\; 13.3 \;\; 14.2 \;\; 15.2) \cdot kip$

$V_{u.Floor.PLh1}^{T} = (3.7 \;\; 4.2 \;\; 4.7 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.6 \;\; 7.1 \;\; 7.6) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^{T} = (8 \;\; 9 \;\; 10 \;\; 11.1 \;\; 12.1 \;\; 13.1 \;\; 14.2 \;\; 15.2 \;\; 16.2) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.2 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^{T} = (4 \;\; 4.5 \;\; 5 \;\; 5.5 \;\; 6 \;\; 6.6 \;\; 7.1 \;\; 7.6 \;\; 8.1) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} = 228.2 \cdot kip \cdot in$ $Mu_{5d.4} = 3.4 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\dfrac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

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$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^{T} = (6.5 \quad 7.4 \quad 8.2 \quad 9.1 \quad 9.9 \quad 10.8 \quad 11.6 \quad 12.5 \quad 13.3)\cdot\%$

$chk_{5d.3} := if(1.0 \geq 2\,max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} = 283.9\cdot kip\cdot in$ $Mu_{5d.5} = 4.2\cdot kip\cdot in$

$chk_{5d.4} := if\left(P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

$\phi P_{5d.5} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.5}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.5}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9\cdot kip$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^{T} = (8.7 \quad 9.8 \quad 10.9 \quad 12.1 \quad 13.2 \quad 14.3 \quad 15.4 \quad 16.6 \quad 17.7)\cdot\%$

$chk_{5d.5} := if(1.0 \geq 2\,max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

$P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} = 324.5\cdot kip\cdot in$ $Mu_{5d.6} = 3.9\cdot kip\cdot in$

$chk_{5d.6} := if\left(P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.6}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.6}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7\cdot kip$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^{T} = (10.8 \quad 12.2 \quad 13.6 \quad 15 \quad 16.4 \quad 17.8 \quad 19.2 \quad 20.6 \quad 22)\cdot\%$

$chk_{5d.7} := if(1.0 \geq 2\,max(Unity_{5d.4}), "OK", "NG")$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w\cdot\rho_{min.5e.1}$ $Amin_{5e.1} = 0.096\cdot\frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^{T} = (1.8 \quad 1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_0} = 9\,ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^{T} = (2.6 \quad 2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3)\cdot kip$

$V_{L.ewh0}^{T} = (0.7 \quad 0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.4)\cdot kip$

$Unity_{5e.1.h0}^{T} = (27.2 \quad 31.2 \quad 35.3 \quad 39.6 \quad 44 \quad 48.5 \quad 53.2 \quad 58.1 \quad 63.1)\cdot\%$

$chk_{5e.1} := if(1.0 \geq max(Unity_{5e.1.h0}), "OK", "NG")$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^{T} = (1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5)$

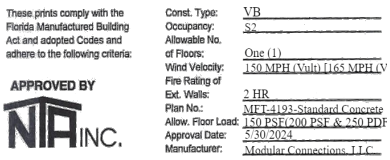
$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_1} = 10\,ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^{T} = (2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.3 \quad 2.2)\cdot kip$

$V_{L.ewh1}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.3 \quad 1.4 \quad 1.5)\cdot kip$

$Unity_{5e.1.h1}^{T} = (29.5 \quad 34 \quad 38.5 \quad 43.3 \quad 48.2 \quad 53.3 \quad 58.6 \quad 64.1 \quad 69.8)\cdot\%$



$chk_{5e.2} := if\left(1.0 \geq max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^{T} = (1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5 \quad 1.5)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_2} = 10.7\,ft \qquad Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^{T} = (2.5 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh2}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^{T} = (30.1 \quad 34.7 \quad 39.4 \quad 44.3 \quad 49.4 \quad 54.7 \quad 60.3 \quad 66 \quad 72.1) \cdot \%$

$chk_{5e.3} := if\left(1.0 \geq max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^{T} = (6.7 \quad 7.6 \quad 8.5 \quad 9.3 \quad 10.2 \quad 11.1 \quad 11.9 \quad 12.8 \quad 13.7) \cdot kip$

$Mu_{5d.4} = 3.35 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \qquad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.4} = 15.7 \qquad d_{5e.4} := \frac{t_w}{2} \qquad d_{5e.4} = 2 \cdot in \qquad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'^{T}_{5e.4} = (0.5 \quad 0.44 \quad 0.4 \quad 0.36 \quad 0.33 \quad 0.3 \quad 0.28 \quad 0.26 \quad 0.24) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \qquad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]\right]$$

$P_{d.5e.4}^{T} = (237.3 \quad 246.3 \quad 253.4 \quad 259.3 \quad 264.1 \quad 268.2 \quad 271.6 \quad 274.6 \quad 277.3) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \qquad Unity_{5e.4}^{T} = (2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.1 \quad 4.4 \quad 4.7 \quad 4.9) \cdot \%$

$chk_{5e.4} := if\left(1.0 \geq max\left(Unity_{5e.4}\right), "OK", "NG"\right)$ $chk_{5e.4} = "OK"$

$P_{5d.5}^{T} = (7.5 \quad 8.4 \quad 9.4 \quad 10.4 \quad 11.3 \quad 12.3 \quad 13.3 \quad 14.2 \quad 15.2) \cdot kip$

$Mu_{5d.5} = 4.2 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \qquad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.5} = 15.7 \qquad d_{5e.5} := \frac{t_w}{2} \qquad d_{5e.5} = 2 \cdot in \qquad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'^{T}_{5e.5} = (0.56 \quad 0.5 \quad 0.45 \quad 0.41 \quad 0.37 \quad 0.34 \quad 0.32 \quad 0.3 \quad 0.28) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}}\right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \qquad x_{5e.5} = 159.1 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$P_{d.5e.5} := x_{5e.5}\cdot\left[\left[-\rho_{5e.5}\cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2\cdot\frac{e'_{5e.5}}{d_{5e.5}}\cdot\rho_{5e.5}\cdot u_{5e.5}}\right]\right]$$

$P_{d.5e.5}^{T} = (227.2 \quad 237.3 \quad 245.4 \quad 252 \quad 257.4 \quad 262 \quad 265.9 \quad 269.3 \quad 272.3)\cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}}$ $\quad Unity_{5e.5}^{T} = (3.3 \quad 3.6 \quad 3.8 \quad 4.1 \quad 4.4 \quad 4.7 \quad 5 \quad 5.3 \quad 5.6)\cdot\%$

$chk_{5e.5} := if\left(1.0 \geq \max\left(Unity_{5e.5}\right), "OK", "NG"\right)$ $\quad chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (8 \quad 9 \quad 10 \quad 11.1 \quad 12.1 \quad 13.1 \quad 14.2 \quad 15.2 \quad 16.2)\cdot kip$

$Mu_{5d.6} = 3.85\cdot kip\cdot in$ $\quad f'_c = 4500\cdot psi$ $\quad \phi_c = 0.65$ $\quad \rho_{5e.6} := \frac{A_{s.4a}\cdot ft}{t_w\cdot b_{eff.5d.6}}$ $\quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85\cdot f'_c}$ $\quad u_{5e.6} = 15.7$ $\quad d_{5e.6} := \frac{t_w}{2}$ $\quad d_{5e.6} = 2\cdot in$ $\quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

$e'^{T}_{5e.6} = (0.48 \quad 0.43 \quad 0.38 \quad 0.35 \quad 0.32 \quad 0.29 \quad 0.27 \quad 0.25 \quad 0.24)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P_{d.5e.6}^{T} = (239.7 \quad 248.4 \quad 255.3 \quad 261 \quad 265.7 \quad 269.6 \quad 273 \quad 275.9 \quad 278.5)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity_{5e.6}^{T} = (3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.6 \quad 4.9 \quad 5.2 \quad 5.5 \quad 5.8)\cdot\%$

$chk_{5e.6} := if\left(1.0 \geq \max\left(Unity_{5e.6}\right), "OK", "NG"\right)$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V_{u.Floor.PLh0}^{T} = (3.4 \quad 3.8 \quad 4.2 \quad 4.7 \quad 5.1 \quad 5.5 \quad 6 \quad 6.4 \quad 6.8)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix} 735.3 \\ 774.6 \\ 800.8 \end{pmatrix}\cdot plf$

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix} 661.7 \\ 697.1 \\ 720.7 \end{pmatrix}\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx}^{T} = (735.3 \quad 774.6 \quad 800.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w_{u.5f.1.DL.mx}^{T} = (661.7 \quad 697.1 \quad 720.7)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix} 693.3 \\ 732.6 \\ 758.8 \end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{\left(H_{wall} - t_{floor}\right)}{3}$

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$w_{5f.1} = \begin{pmatrix} 2.8 \\ 3.1 \\ 3.3 \end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix} w_{DL.5f.1.mx} \\ -w_{DL.5f.1.mx} \end{pmatrix}}{No_{plates}} = \begin{bmatrix} \begin{pmatrix} 735.25 \\ 774.58 \\ 800.81 \end{pmatrix} \\ \begin{pmatrix} -735.25 \\ -774.58 \\ -800.81 \end{pmatrix} \end{bmatrix} \cdot plf$

$w_{up} := \frac{\left|P_{wind_3}\right| \cdot W_{ew}}{2}$ $w_{up} = 311.9 \cdot plf$ $w_{u.up} := w_{up}$ $w_{u.up} = 311.9 \cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h0} = (5.8 \quad 6.3 \quad 6.7 \quad 7.1 \quad 7.6 \quad 8 \quad 8.4 \quad 8.9 \quad 9.3) \cdot kip$

$P_{5f.2.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h1} = (6.1 \quad 6.5 \quad 6.9 \quad 7.4 \quad 7.8 \quad 8.2 \quad 8.7 \quad 9.1 \quad 9.5) \cdot kip$

$P_{5f.2.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h2} = (6.2 \quad 6.6 \quad 7.1 \quad 7.5 \quad 8 \quad 8.4 \quad 8.8 \quad 9.3 \quad 9.7) \cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

Tension Side

$P_{5f.3.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h0} = (-0.9 \quad -1.3 \quad -1.8 \quad -2.2 \quad -2.6 \quad -3.1 \quad -3.5 \quad -3.9 \quad -4.4) \cdot kip$

$P_{5f.3.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h1} = (-0.7 \quad -1.1 \quad -1.5 \quad -2 \quad -2.4 \quad -2.8 \quad -3.3 \quad -3.7 \quad -4.1) \cdot kip$

$P_{5f.3.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h2} = (-0.5 \quad -0.9 \quad -1.4 \quad -1.8 \quad -2.2 \quad -2.7 \quad -3.1 \quad -3.6 \quad -4) \cdot kip$

$W_{u.5f.1} := \left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| = 53.3 \cdot psf$ $Mu_{5f.1} := \frac{W_{u.5f.1} \cdot (H_E - 8in)^2}{8} = \begin{pmatrix} 6.3 \\ 7.8 \\ 8.9 \end{pmatrix} \cdot \frac{kip \cdot in}{ft}$

$d_{5.4} := 0.5 \cdot t_w$ $d_{5.4} = 2 \cdot in$ $x_{5f.2} := 0.00349$ Initial Guess for reinforcement ratio

$$f(x_{5f.2}) := \frac{\frac{\max(Mu_{5f.1})}{\left(\frac{kip \cdot ft}{ft}\right)}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d_{5.4}}{in}\right)^2 \cdot (1 - 8.85 \cdot x_{5f.2})} - x_{5f.2} \quad \rho_{5f.2} := \frac{root(f(x_{5f.2}), x_{5f.2})}{ft} \quad \rho_{5f.2} = 0.00355 \cdot \frac{1}{ft}$$

Reinforcement Ratio Required for out of plane Wind

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$A_{s.req1.5f.2.} := \rho_{5f.2} \cdot 12in \cdot d_{5.4}$ $A_{s.req1.5f.2.} = 0.085 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for out of plane Wind

$$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9 \cdot f_y \cdot \frac{W_{ew}}{2}}\right)$$

$A_{s.req2.5f.2} = 0.01389 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for Tension Force from Uplift

$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2}$ $A_{req.5f.2} = 0.099 \cdot \frac{in^2}{ft}$ Total Area of Reinforcement Required

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ Area of Steel Provided

$chk_{5.8} := if\left(A_{req.5f.2} \le A_{s.4a}, "OK", "NG"\right)$ $chk_{5.8} = "OK"$

Check P-Δ Effects

$E_{c.spec} = 2991 \cdot ksi$ $I_{g.5g.1} := \frac{1}{12} \cdot 12in \cdot \left(t_w\right)^3$ $I_{g.5g.1} = 64 \cdot in^4$

$P_{cr.5g.1} := \frac{\pi^2 \cdot E_{c.spec} \cdot I_{g.5g.1}}{H_E^2}$ $P_{cr.5g.1} = \begin{pmatrix} 143.5 \\ 117.6 \\ 104.1 \end{pmatrix} \cdot kip$ $Ma_{g.2} := \frac{\left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| \cdot \left(H_{wall}\right)^2}{8} \cdot 12in$ $Ma_{g.2} = \begin{pmatrix} 539.3 \\ 665.9 \\ 757.6 \end{pmatrix} \cdot lb \cdot ft$

$Mb_{5g.2} := Ma_{g.2}$ $Mb_{5g.2} = \begin{pmatrix} 0.5 \\ 0.7 \\ 0.8 \end{pmatrix} \cdot kip \cdot ft$ $C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}}$ $C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}}$$

$\delta_{ns.5.6} = \begin{pmatrix} 1.2 \\ 1.2 \\ 1.2 \end{pmatrix}$

$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix}$ $Ma_{g.3} = \begin{pmatrix} 623.6 \\ 797.3 \\ 931.1 \end{pmatrix} \cdot lb \cdot ft$ $f_r = 427.6 \cdot psi$ $M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}}$ $M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking Occurs", "Cracking Doesn't Occur"\right)$ $chk_{5.9} = "Cracking Doesn't Occur"$

$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}}$ $\Delta_{5g.1} = \begin{pmatrix} 0.18 \\ 0.23 \\ 0.27 \end{pmatrix} \cdot in$ $Mu_{5.5} := Mu_{5f.1} \cdot ft$ $Mu_{5.5} = \begin{pmatrix} 6.3 \\ 7.8 \\ 8.9 \end{pmatrix} \cdot kip \cdot in$

$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$

$Mu_{5.6} = (10 \quad 10.2 \quad 10.3 \quad 10.5 \quad 10.6 \quad 10.7 \quad 10.9 \quad 11 \quad 11.2) \cdot kip \cdot in$

$P_{5.8} := Embed_{5f.1}$

$P_{5.8} = (3.4 \quad 3.8 \quad 4.2 \quad 4.7 \quad 5.1 \quad 5.5 \quad 6 \quad 6.4 \quad 6.8) \cdot kip$

$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}}$ $e`_{5.6} = (3 \quad 2.7 \quad 2.4 \quad 2.2 \quad 2.1 \quad 1.9 \quad 1.8 \quad 1.7 \quad 1.6) \cdot in$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$u_{5.} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f'_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

$$\rho_{5.8} := \frac{A_{s.4a}}{\max(w_{5f.1}) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max(e`_{5.6})}{d_{5.4}} + \sqrt{\left(1 - \frac{\max(e`_{5.6})}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max(e`_{5.6}) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 39.7 \\ 44.4 \\ 47.6 \end{pmatrix} \cdot kip$$

$$chk_{5.10} := \text{if}(\max(P_{5.8}) \le \min(\phi P_{5.6}), \text{"OK"}, \text{"NG"}) \qquad chk_{5.10} = \text{"OK"}$$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"

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Fire Rating of Ext. Walls:	2 HR
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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 41.1 \\ 40.4 \\ 39.8 \\ 39.4 \\ 39.1 \\ 38.8 \\ 38.6 \\ 37.8 \\ 37.7 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 12 ' "} \\ \text{"Min Flr Uniform Weight, for L = 14 ' "} \\ \text{"Min Flr Uniform Weight, for L = 16 ' "} \\ \text{"Min Flr Uniform Weight, for L = 18 ' "} \\ \text{"Min Flr Uniform Weight, for L = 20 ' "} \\ \text{"Min Flr Uniform Weight, for L = 22 ' "} \\ \text{"Min Flr Uniform Weight, for L = 24 ' "} \\ \text{"Min Flr Uniform Weight, for L = 26 ' "} \\ \text{"Min Flr Uniform Weight, for L = 28 ' "} \end{pmatrix}$$

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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$$wg_{f.mx} = \begin{pmatrix} 45.1 \\ 44.4 \\ 43.8 \\ 43.4 \\ 43.1 \\ 42.8 \\ 42.6 \\ 41.8 \\ 41.7 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf \quad \text{Wall Weight} \qquad w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf \quad \text{Wall Misc. Weight}$$

$DL_{r.mx} = 63.5 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 56.5 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -53.48 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 46 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.2$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

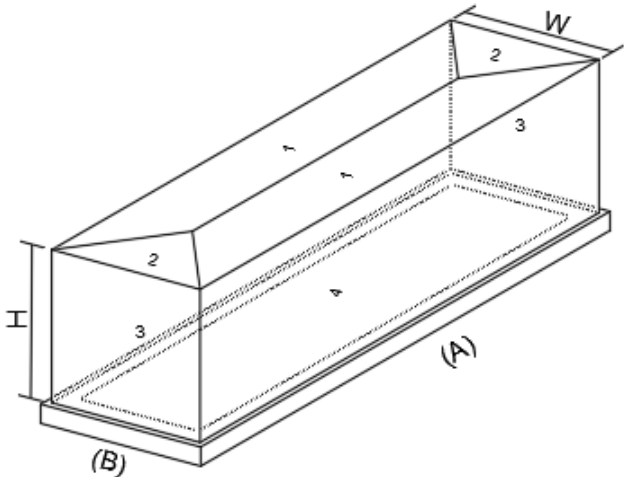
$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.16$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\ ft^2$

$A_f^T = (123.74 \quad 145.65 \quad 167.57 \quad 189.49 \quad 211.4 \quad 233.32 \quad 255.24 \quad 277.15 \quad 299.07)\ ft^2$

$L_{wall}^T = (45.92 \quad 49.92 \quad 53.92 \quad 57.92 \quad 61.92 \quad 65.92 \quad 69.92 \quad 73.92 \quad 77.92)\ ft$



$$DL_{A.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) \ldots + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 45.73\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 39.77\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\left[\begin{array}{l}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor} \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{array}\right]}{\begin{array}{l}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right] \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{array}}\right] \qquad \begin{array}{l} z_{mx} = 67.38\cdot in \\ z_{mx} = 5.61\cdot ft\end{array}$$

$ftg_{exp} := 36in$ Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.73\,ft$ Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\,ft$ Moment Arm for Floor Level

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$DL_{A.f0.mx}^{T} = (1108.4 \;\; 1104.3 \;\; 1101.3 \;\; 1098.9 \;\; 1097.1 \;\; 1095.6 \;\; 1094.3 \;\; 1090 \;\; 1089.3)\cdot plf$

$DL_{A.f0.mn}^{T} = (1284.5 \;\; 1225.3 \;\; 1181.6 \;\; 1148.1 \;\; 1121.4 \;\; 1099.8 \;\; 1081.9 \;\; 1063.6 \;\; 1050.9)\cdot plf$

$DL_{B.f0.mx}^{T} = (854.3 \;\; 850.1 \;\; 847.1 \;\; 844.8 \;\; 842.9 \;\; 841.4 \;\; 840.2 \;\; 835.8 \;\; 835.2)\cdot plf$

$DL_{B.f0.mn}^{T} = (1058.3 \;\; 999.1 \;\; 955.5 \;\; 921.9 \;\; 895.3 \;\; 873.7 \;\; 855.8 \;\; 837.4 \;\; 824.8)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1503 \;\; 1499 \;\; 1496 \;\; 1494 \;\; 1492 \;\; 1491 \;\; 1489 \;\; 1485 \;\; 1484)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (1284 \;\; 1225 \;\; 1182 \;\; 1148 \;\; 1121 \;\; 1100 \;\; 1082 \;\; 1064 \;\; 1051)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (821 \;\; 820 \;\; 819 \;\; 818 \;\; 817 \;\; 817 \;\; 816 \;\; 815 \;\; 815)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (592 \;\; 591 \;\; 590 \;\; 589 \;\; 589 \;\; 588 \;\; 588 \;\; 586 \;\; 586)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C}$ $\qquad$ $E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

$E_{O.A.E.f0.C}^{T} = (277.9 \;\; 277.2 \;\; 276.6 \;\; 276.2 \;\; 275.8 \;\; 275.6 \;\; 275.3 \;\; 274.5 \;\; 274.4)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-277.9 \;\; -277.2 \;\; -276.6 \;\; -276.2 \;\; -275.8 \;\; -275.6 \;\; -275.3 \;\; -274.5 \;\; -274.4)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (147.3 \;\; 124.9 \;\; 108.5 \;\; 95.8 \;\; 85.8 \;\; 77.7 \;\; 71 \;\; 65.3 \;\; 60.5)\cdot plf$

$E_{O.B.E.f0.T}^{T} = (-147.3 \;\; -124.9 \;\; -108.5 \;\; -95.8 \;\; -85.8 \;\; -77.7 \;\; -71 \;\; -65.3 \;\; -60.5)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (265.3 \;\; 264.5 \;\; 263.9 \;\; 263.5 \;\; 263.2 \;\; 262.9 \;\; 262.7 \;\; 261.9 \;\; 261.7)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-265.3 \;\; -264.5 \;\; -263.9 \;\; -263.5 \;\; -263.2 \;\; -262.9 \;\; -262.7 \;\; -261.9 \;\; -261.7)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (143.2 \;\; 121.4 \;\; 105.4 \;\; 93.1 \;\; 83.4 \;\; 75.5 \;\; 69 \;\; 63.4 \;\; 58.8)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-143.2 \;\; -121.4 \;\; -105.4 \;\; -93.1 \;\; -83.4 \;\; -75.5 \;\; -69 \;\; -63.4 \;\; -58.8)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$E_{O.A.E.f2.C}^{T} = (252.6 \;\; 251.8 \;\; 251.3 \;\; 250.9 \;\; 250.5 \;\; 250.2 \;\; 250 \;\; 249.2 \;\; 249.1)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-252.6 \;\; -251.8 \;\; -251.3 \;\; -250.9 \;\; -250.5 \;\; -250.2 \;\; -250 \;\; -249.2 \;\; -249.1)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (139.1 \;\; 118 \;\; 102.4 \;\; 90.5 \;\; 81 \;\; 73.4 \;\; 67 \;\; 61.6 \;\; 57.1)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-139.1 \;\; -118 \;\; -102.4 \;\; -90.5 \;\; -81 \;\; -73.4 \;\; -67 \;\; -61.6 \;\; -57.1)\cdot plf$

$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$

$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$

$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$

$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$

$E_{8n.A.f0}^{T} = (819.9 \;\; 817.7 \;\; 816 \;\; 814.8 \;\; 813.8 \;\; 812.9 \;\; 812.3 \;\; 809.9 \;\; 809.5)\cdot plf$

$E_{8n.A.f0}^{T} = (-226.5 \;\; -228.1 \;\; -229.3 \;\; -230.2 \;\; -230.9 \;\; -231.5 \;\; -232 \;\; -231.9 \;\; -232.3)\cdot plf$

$E_{8n.B.f0}^{T} = (443.3 \;\; 420.4 \;\; 403.6 \;\; 390.7 \;\; 380.4 \;\; 372.2 \;\; 365.3 \;\; 359 \;\; 354.2)\cdot plf$

$E_{8n.B.f0}^{T} = (-123.6 \;\; -101.2 \;\; -84.8 \;\; -72.2 \;\; -62.2 \;\; -54.1 \;\; -47.5 \;\; -41.8 \;\; -37)\cdot plf$

$E_{8n.A.f1}^{T} = (782.6 \;\; 780.3 \;\; 778.7 \;\; 777.4 \;\; 776.4 \;\; 775.6 \;\; 774.9 \;\; 772.5 \;\; 772.2)\cdot plf$

$E_{8n.B.f1}^{T} = (431 \;\; 408.7 \;\; 392.3 \;\; 379.8 \;\; 369.8 \;\; 361.8 \;\; 355.1 \;\; 349 \;\; 344.2)\cdot plf$

$E_{8n.A.f1}^{T} = (-210.3 \;\; -209.8 \;\; -209.3 \;\; -209 \;\; -208.7 \;\; -208.5 \;\; -208.3 \;\; -207.7 \;\; -207.6)\cdot plf$

$E_{8n.B.f1}^{T} = (-112.1 \;\; -90.4 \;\; -74.4 \;\; -62.1 \;\; -52.5 \;\; -44.6 \;\; -38.1 \;\; -32.6 \;\; -27.9)\cdot plf$

$E_{8n.B.f2}^{T} = (418.6 \;\; 397 \;\; 381.1 \;\; 368.9 \;\; 359.2 \;\; 351.4 \;\; 344.9 \;\; 338.9 \;\; 334.3)\cdot plf$

$E_{8n.A.f2}^{T} = (-200.4 \;\; -199.8 \;\; -199.4 \;\; -199.1 \;\; -198.8 \;\; -198.6 \;\; -198.4 \;\; -197.8 \;\; -197.7)\cdot plf$

$E_{8n.A.f2}^{T} = (745.2 \;\; 743 \;\; 741.3 \;\; 740.1 \;\; 739 \;\; 738.2 \;\; 737.5 \;\; 735.2 \;\; 734.8)\cdot plf$

$E_{8n.B.f2}^{T} = (-108.9 \;\; -87.8 \;\; -72.3 \;\; -60.4 \;\; -51 \;\; -43.3 \;\; -37 \;\; -31.7 \;\; -27.1)\cdot plf$

$LL_{r.A.} := \frac{W_r}{2}\cdot(w_{llr.2})$ $\quad LL_{r.A.} = 210\cdot plf$ $\quad LL_{r.B.} := \frac{W_r}{6}\cdot(w_{llr.2})$ $\quad LL_{r.B.} = 70\cdot plf$

$LL_{A.f0} := \frac{W_f}{2}\cdot(w_{llf.0})$ $\quad LL_{A.f0} = 1369.79\cdot plf$ $\quad LL_{B.f0} := s_{t.rib}\cdot(w_{llf.0})$ $\quad LL_{B.f0} = 500\cdot plf$

$LL_{A.f1} := \frac{W_f}{2}\cdot(w_{llf.1})$ $\quad LL_{A.f1} = 1095.83\cdot plf$ $\quad LL_{B.f1} := s_{t.rib}\cdot(w_{llf.1})$ $\quad LL_{B.f1} = 400\cdot plf$

$LL_{A.f2} := \frac{W_f}{2}\cdot(w_{llf.2})$ $\quad LL_{A.f2} = 821.88\cdot plf$ $\quad LL_{B.f2} := s_{t.rib}\cdot(w_{llf.2})$ $\quad LL_{B.f2} = 300\cdot plf$

$SL_A := \frac{W_r}{2}\cdot(p_f)$ $\quad SL_A = 190.51\cdot plf$ $\quad SL_B := \frac{W_r}{6}\cdot(p_f)$ $\quad SL_B = 63.5\cdot plf$

$W_{up.A} := \frac{W_r}{2}\cdot P_{wind_3}$ $\quad W_{up.A} = -320.86\cdot plf$ $\quad W_{up.B} := \frac{W_r}{6}\cdot P_{wind_3}$ $\quad W_{up.B} = -106.95\cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $\quad W_{OT.A.C} = 618.5\cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $\quad W_{OT.B.C} = 600.2\cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $\quad W_{OT.A.T} = -618.5\cdot plf$ $\quad W_{OT.B.T} := -W_{OT.B.C}$ $\quad W_{OT.B.T} = -600.2\cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $\quad W_{A.mx} = 297.6\cdot plf$ $\quad W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $\quad W_{B.mx} = 493.3\cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $\quad W_{A.mn} = -939.4\cdot plf$ $\quad W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $\quad W_{B.mn} = -707.2\cdot plf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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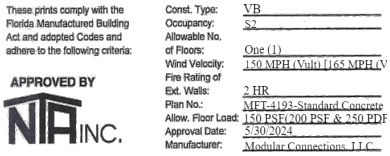
b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \qquad Load_{f0} = \begin{pmatrix} 1108.43 \\ 1050.93 \\ 1369.79 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 819.94 \\ -232.3 \end{pmatrix} \cdot plf$$



$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f0} := asd \cdot Load_{f0} \qquad LC_{8.f0} = \begin{pmatrix} 2478 \\ 1318 \\ 2346 \\ 1287 \\ 545 \\ 2464 \\ 1536 \\ 809 \\ 67 \\ 1682 \\ 946 \\ 2709 \\ 2157 \\ 1205 \\ 468 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 13 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 14 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f0} := \max(15in, w_{8.f0.min})$ $Fdn_{W.f0} = 15 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 15 \cdot in$

consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \qquad Load_{f1} = \begin{pmatrix} 1108.43 \\ 1050.93 \\ 1095.83 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 782.58 \\ -210.35 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f1} := asd \cdot Load_{f1} \qquad LC_{8.f1} = \begin{pmatrix} 2204 \\ 1318 \\ 2140 \\ 1287 \\ 545 \\ 2259 \\ 1331 \\ 809 \\ 67 \\ 1656 \\ 961 \\ 2484 \\ 1963 \\ 1178 \\ 483 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 11.92 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 12 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f1} := \max(14in, w_{8.f1.min}) \qquad Fdn_{W.f1} = 14 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \qquad Load_{f2} = \begin{pmatrix} 1108.43 \\ 1050.93 \\ 821.88 \\ 0 \\ 210 \\ 190.51 \\ 297.65 \\ -939.36 \\ 745.23 \\ -200.43 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f2} := asd \cdot Load_{f2} \qquad LC_{8.f2} = \begin{pmatrix} 1930 \\ 1318 \\ 1935 \\ 1287 \\ 545 \\ 2053 \\ 1125 \\ 809 \\ 67 \\ 1630 \\ 968 \\ 2259 \\ 1762 \\ 1152 \\ 490 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 10.84 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 11 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f2} := \max(12in, w_{8.f2.min}) \qquad Fdn_{W.f2} = 12 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 12 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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c. Slab on Grade Foundation

1.) Criteria: use slab on grade, $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.long}$ $A_{s.long.prov} = 0.39 \cdot in^2$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if(A_{s.long.prov} \geq A_{s.long}, "OK", "NG")$ $chk_{8.1} = "OK"$

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APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp}$ $A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4$ $Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.temp}$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG")$ $chk_{8.2} = "OK"$

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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 56.542 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 63.5 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 80 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 46 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 35.8 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 32.3 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 53.5 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 3.01 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 3.012 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 7.229 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.027 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.027 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.31 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.31 \cdot klf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SEISMIC LOAD PARAMETERS: *For worst case seismic load in E-W direction, look at longest, widest shelter.*

Seismic Coefficient

$C_s = 0.2$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 28.4\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 26.02\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 27.08\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 22.85\cdot kip$

Total Seismic Shear at Roof

$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$ $V_{smaxW\_rf} = 8.401\cdot kip$

$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$ $V_{sminW\_rf} = 7.501\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$ $v_{EW\_smaxW} = 0.35\cdot klf$

$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$ $v_{EW\_sminW} = 0.134\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$ $v_{NS\_smaxW} = 0.36\cdot klf$

$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$ $v_{NS\_sminW} = 0.321\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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Occupancy: S2
Allowable No. of Floors: One (1)
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Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

k_c — Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ — Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ — Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ — Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} — Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} — Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ — Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ — Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ — Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix}$ = ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ — Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.048 \cdot klf$ — Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.36 \cdot klf$ — Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.16 \cdot klf$ — Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.212 \cdot klf$ — Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.094 \cdot klf$ — Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.31 \cdot klf$ — Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 7$

$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1}$ $s_{REW} = 18.67\cdot in$

$s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$ $s_{EWR} = 19.33\cdot in$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, \min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, \min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$	$Vu_{o.s.REW} = 0.026\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$	$Vu_{i.s.REW} = 0.653\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$	$Vu_{o.w.REW} = 0.171\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$	$Vu_{i.w.REW} = 0.562\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.REW} := F_{7.1}\cdot\max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$	$Tu_{T.REW} = 0\cdot kip$	0.9DL + 1.0W	T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$	$Vu_{o.s.EWR} = 0.027\cdot kip$	0.3E	out-of-plane V, seismic
$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$	$Vu_{i.s.EWR} = 0.677\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{o.w.EWR} := F_{7.2}\cdot\max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$	$Vu_{o.w.EWR} = 0.177\cdot kip$	1.0W	out-of-plane V, wall suction
$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$	$Vu_{i.w.EWR} = 0.582\cdot kip$	1.0W	in-plane V roof diaphragm
$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$	$Tu_{T.EWR} = 0.177\cdot kip$	0.9DL + 1.0W	T, roof uplift

▶ Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 5\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 5 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 112 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 5.72 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 5 \cdot in$ $h_{min.7.1a} = 5 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 134 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 3.76 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 4.287 \\ 2.816 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min((0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1}))$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.238 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.239 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.026$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.289 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.296 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.1} = 0.198 \cdot in$$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.2} = 0.186 \cdot in$$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2} \qquad l_{w\_Rqrd.REW.3} = 2.5 \cdot in$$

$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right)$ $l_{w\_reqd.REW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.048\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.35\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.321\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.381\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.188\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.027\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 48\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.3} := \text{if}(s_{L.7.3} = 0in, s_{w.7.3}, \text{if}(s_{w.7.3} = 0in, s_{L.7.3}, \min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := \text{if}(s_{L.7.4} = 0in, s_{w.7.4}, \text{if}(s_{w.7.4} = 0in, s_{L.7.4}, \min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.226\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.49\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 0.878\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.126\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.226\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.49\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 0.878\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.126\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.28 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.338 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.272 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.319 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.158\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.26\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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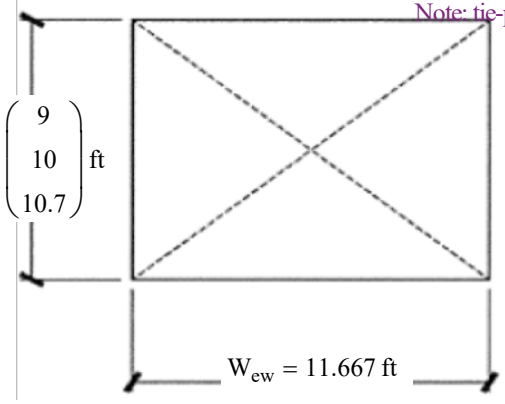
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 6$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}$ ft

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (16.4 \quad 18.8 \quad 20.4)\cdot in$ $s_{SWEW}^{T} = (16.4 \quad 18.8 \quad 20.4)\cdot in$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6\,ft$ Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 5.275\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{..}\cdot L_{..}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (1.319 \quad 1.135 \quad 0.996 \quad 0.888 \quad 0.8 \quad 0.729 \quad 0.669 \quad 0.618 \quad 0.574)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.007\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 1.13\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10\,ft$

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

$M_{ot\_w.EWSW}^{T} = (33 \quad 35.9 \quad 38.9 \quad 41.9 \quad 44.9 \quad 47.9 \quad 50.9 \quad 53.9 \quad 56.9)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r + \frac{\max\left[(\omega_{wall})\cdot\left[\max(H_{wall})\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (51.9 \quad 55.9 \quad 59.9 \quad 63.9 \quad 67.9 \quad 72 \quad 76 \quad 80 \quad 84)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (45.5 \quad 50.7 \quad 56 \quad 61.3 \quad 66.6 \quad 71.9 \quad 77.2 \quad 82.5 \quad 87.8)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (-7.96 \quad -9.73 \quad -11.5 \quad -13.27 \quad -15.05 \quad -16.82 \quad -18.59 \quad -20.36 \quad -22.13)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (6.43 \quad 5.15 \quad 3.88 \quad 2.61 \quad 1.33 \quad 0.06 \quad -1.21 \quad -2.49 \quad -3.76)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net})$ $M_{ot.EWSW} = 6.4\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0.55\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := if\left(s_{L.7.5} = 0in, s_{w.7.5}, if\left(s_{w.7.5} = 0in, s_{L.7.5}, \min\left(s_{L.7.5}, s_{w.7.5}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := if\left(s_{L.7.6} = 0in, s_{w.7.6}, if\left(s_{w.7.6} = 0in, s_{L.7.6}, \min\left(s_{L.7.6}, s_{w.7.6}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0.107\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.39\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.39\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wal (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0.107 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.39 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.39 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.091$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.189$ < 1.0

$Result_{EWSW} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW} =$ "not required" < 1.2

$chk_{6.EWSW} =$ "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.092$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.302$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.165 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.165 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right)$ $l_{w\_reqd.EWSW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.68 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.247 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.024 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.36 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.31 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.165 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.094 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\,ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

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$M_{ot.s.FEW}^{T} = (36.4 \quad 42.3 \quad 48.3 \quad 54.2 \quad 60.2 \quad 66.2 \quad 72.1 \quad 78.1 \quad 84)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No.
of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of
Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix} 0.5\cdot(L_0)^2 \\ 0.5\cdot(L_1)^2 \\ 0.5\cdot(L_2)^2 \\ 0.5\cdot(L_3)^2 \\ 0.5\cdot(L_4)^2 \\ 0.5\cdot(L_5)^2 \\ 0.5\cdot(L_6)^2 \\ 0.5\cdot(L_7)^2 \\ 0.5\cdot(L_8)^2 \end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (93.3 \quad 114.4 \quad 138.7 \quad 166.3 \quad 197.1 \quad 231.1 \quad 268.5 \quad 309 \quad 352.8)\cdot kip\cdot ft$

$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$

OT Moment, resisting

$M_{res.FEW}^{T} = (293.2 \quad 342.1 \quad 391 \quad 439.8 \quad 488.7 \quad 537.6 \quad 586.5 \quad 635.3 \quad 684.2)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-170.6 \quad -193.5 \quad -213.2 \quad -229.6 \quad -242.8 \quad -252.7 \quad -259.4 \quad -262.8 \quad -263)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = (-256.8 \quad -299.8 \quad -342.7 \quad -385.6 \quad -428.5 \quad -471.4 \quad -514.4 \quad -557.3 \quad -600.2)\cdot kip\cdot ft$

$M_{ot.FEW} := \max(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net})$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right]$ $\quad v_{ot.w.FEW} = 0\cdot kip$

$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right)$ $\quad v_{ot.s.FEW} = 0\cdot kip$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor

$Plate_{7.8}$ = "#2" at inside face of end wall

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

No. of Plates & maximum plate spacing $No_{pl.FEW} = 8$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \qquad s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1}$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$s_{F\_EW} = 13.64 \cdot in$ $s_{EW\_F} = 14.86 \cdot in$

$s_{7.7} := if\left(s_{L.7.7} = 0in, s_{w.7.7}, if\left(s_{w.7.7} = 0in, s_{L.7.7}, min\left(s_{L.7.7}, s_{w.7.7}\right)\right)\right) = 3 \cdot in$ $s_{7.7} = 3 \cdot in$

$$F_{7.7} := 0.5 + \frac{0.5s_{7.7} + 2e_{PL.F\_EW}}{s_{7.7}} \qquad F_{7.7} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.7

$s_{7.8} := if\left(s_{L.7.8} = 0in, s_{w.7.8}, if\left(s_{w.7.8} = 0in, s_{L.7.8}, min\left(s_{L.7.8}, s_{w.7.8}\right)\right)\right) = 3 \cdot in$ $s_{7.7} = 3 \cdot in$

$$F_{7.8} := 0.5 + \frac{0.5s_{7.8} + 2e_{PL.F\_EW}}{s_{7.8}} \qquad F_{7.8} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.8

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7} \cdot \left(1.0v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{max\left(H_{wall}\right)}{4} \cdot W_{ew}\right)$$ $Vu_{i.s.FEW} = 0.857 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7} \cdot 1.0v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{i.w.FEW} = 0.527 \cdot kip$ 1.0W in-plane V, wind roof diaphragm

$Tu_{po.s1.FEW} := F_{7.7} \cdot 0.3C_s \cdot \left(t_{o.s.FEW} \cdot W_{ew}\right)$ $Tu_{po.s1.FEW} = 0.02 \cdot kip$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$$ $Tu_{po.w.FEW} = 0.32 \cdot kip$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7} \cdot \left(0.9v_{DL.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}}$$ $Vu_{o.9dl.FEW} = 1.041 \cdot kip$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$$ $V_{ot.w.FEW} = 0 \cdot kip$ 1.0W

$$V_{ot.s.FEW} := F_{7.7} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$$ $V_{ot.s.FEW} = 0 \cdot kip$ 1.0E

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$Vu_{o.wup.FEW} := F_{7.7} \cdot max\left[\left(1.0v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip\right]$$

$Vu_{o.wup.FEW} = 0 \cdot kip$ 0.9D+1.0W

$Vu_{o.sup.FEW} := F_{7.7} \cdot max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$ Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.sup.FEW} = 0 \cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$$

$Vu_{o.12D.16L.FEW} = 2.06 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

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Factored loads - Floor - Endwall (Floor Wall Plates)

$Tu_{po.w.EWF} := F_{7.8} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Tu_{po.w.EWF} = 0.32 \cdot kip$ pullout, wind suction at bot of wall and bot half of two sides of wall

$Tu_{po.s2.EWF} := F_{7.8} \cdot 0.3 C_s \cdot (t_{o.s.FEW} \cdot W_{ew})$ $Tu_{po.s2.EWF} = 0.02 \cdot kip$ 0.3E pullout, seismic

$Vu_{i.s.EWF} := F_{7.8} \cdot \left(1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{\max(H_{wall})}{4} \cdot W_{ew} \right)$

$Vu_{i.s.EWF} = 0.857 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w2.EWF} := F_{7.8} \cdot 1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{i.w2.EWF} = 0.527 \cdot kip$ 1.0W in-plane V, wind roof diaphragm

$Vu_{o.9dl.EWF} := F_{7.8} \cdot (0.9 v_{DL.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{o.9dl.EWF} = 1.041 \cdot kip$ 0.9D Shear from DL only

$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - (1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 0.933 \cdot kip$ 0.9D+1.0W Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[(1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

$Vu_{o.sdn.EWF} := F_{7.8} \cdot (Vu_{o.9dl.FEW} + V_{ot.s.FEW})$ $Vu_{o.sdn.EWF} = 1.214 \cdot kip$ 0.9D+1.0E Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot \left[1.2(v_{DL.FEW}) + 1.6 v_{LL.FEW} \right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 2.06 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$
$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75\cdot V_{cbg\_op.7.7} \\ 0.75\cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$$\phi P_{pl.F\_EW} = \min\left(\left(0.75\cdot N_{sa.7.7} \quad 0.75\cdot N_{cbg.7.7} \quad 0.7\cdot N_{pn.7.7}\right)\right) \qquad \phi P_{pl.F\_EW} = 8.935\cdot kip$$

Plate Summary

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.036 \quad \text{or} \quad \left[\frac{Vu_{o.wup.FEW}}{0.75\cdot V_{cbg\_op.7.7}} + \frac{Vu_{i.w.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.108 \quad < 1.0$$

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.036 \quad \text{or} \quad \left[\frac{Vu_{o.sup.FEW}}{\left(0.75\cdot V_{cbg\_op.7.7}\right)} + \frac{Vu_{i.s.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.175 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.606 \quad < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8} \cdot A_{SE.7.8} \cdot f_{uta}$ $V_{sa.7.8} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719 \cdot in$ $h_{min.7.8} = 4 \cdot in$ $c_{a1\_op.7.8} = 6.5 \cdot in$ $c_{a2\_op.7.8} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90 \cdot in^2$ $A_{Vco\_op.7.8} = 190 \cdot in^2$ $e'_{V.7.8} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0 \cdot in$ $s2_{7.8a} = 3 \cdot in$ $l_{e.7.8} = 1.719 \cdot in$ $h_{ef.7.8} = 1.719 \cdot in$ $h_{a.7.8} = 4 \cdot in$ $h_{min.7.8a} = 4 \cdot in$

$c_{a1\_ip.7.8} = 13.5 \cdot in$ $c_{a2\_ip.7.8} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107 \cdot in^2$ $A_{Vco\_ip.7.8} = 820 \cdot in^2$ $e'_{V.7.8a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8} \cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75 \cdot V_{cbg\_op.7.8} \\ 0.75 \cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\left(0.75 \cdot N_{sa.7.8} \quad 0.75 \cdot N_{cbg.7.8} \quad 0.7 \cdot N_{pn.7.8}\right)\right)$ $\phi P_{pl.EW\_F} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.005 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.734 \quad < 1.0$$

$$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.074 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.506 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75 \cdot V_{cbg\_op.7.8}}\right) = 0.601 \quad < 1.0$$

$Result_{EW\_F}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F}$ = "not required" < 1.2

$chk_{6.EW\_F}$ = "OK"

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Weld Between $Plate_{7.7}$ = "#1B" and $Plate_{7.8}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.1} = 0.181\cdot in$$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.2} = 0.251\cdot in$$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.3} = 0.328\cdot in$$

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^2 + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^2 + Vu_{i.s.EWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.435\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.435\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$$l_{w\_reqd.EWSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $\quad w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $\quad v_{dl.FSW} = 0.865\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $\quad v_{ll.FSW} = 0.48\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $\quad t_{o.s.FSW} = 0.048\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $\quad v_{i.s.FSW} = 0.35\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $\quad v_{i.w.SW} = 0.0269\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $\quad v_{up.w.FSW} = 0.321\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $\quad t_{po.w.FSW} = 0.172\cdot klf$ Wind load suction on bottom of side wall

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^{T} = (93.3 \quad 114.4 \quad 138.7 \quad 166.3 \quad 197.1 \quad 231.1 \quad 268.5 \quad 309 \quad 352.8)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^{T} = (36.4 \quad 42.3 \quad 48.3 \quad 54.2 \quad 60.2 \quad 66.2 \quad 72.1 \quad 78.1 \quad 84)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^{T} = (293.2 \quad 342.1 \quad 391 \quad 439.8 \quad 488.7 \quad 537.6 \quad 586.5 \quad 635.3 \quad 684.2)\cdot kip\cdot ft$

$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right)$ $v_{ot.w.FSW} = 0\cdot kip$

$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right)$ $v_{ot.s.FSW} = 0\cdot kip$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9} = "\#1B"$ at top of floor $Plate_{7.10} = "\#2"$ at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 18\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.9} := if\left(s_{L.7.9} = 0in, s_{w.7.9}, if\left(s_{w.7.9} = 0in, s_{L.7.9}, \min\left(s_{L.7.9}, s_{w.7.9}\right)\right)\right) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}}$ $F_{7.9} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.9

$s_{7.10} := if\left(s_{L.7.10} = 0in, s_{w.7.10}, if\left(s_{w.7.10} = 0in, s_{L.7.10}, \min\left(s_{L.7.10}, s_{w.7.10}\right)\right)\right) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}}$ $F_{7.10} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$	$Tu_{o.w.FSW} = 0.301\cdot kip$	1.0W	T, wall suction
$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$	$Tu_{o.s.FSW} = 0.085\cdot kip$	0.3W	T, pullout seismic
$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$	$Vu_{i.s.FSW} = 0.184\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$	$Vu_{i.w.FSW} = 0.047\cdot kip$	1.0W	in-plane V roof diaphragm
$Vu_{o\_09dl.FSW} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$	$Vu_{o\_09dl.FSW} = 1.167\cdot kip$	0.9D	Shear from DL only
$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$	$V_{ot\_w.FSW} = 0\cdot kip$	1.0W	Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.
$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$	$V_{ot\_s.FSW} = 0\cdot kip$		

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.FSW} \qquad +-V_{ot\_w.FSW}$$

$Vu_{o.w.FSW} = 1.924\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$

$Vu_{o.s.FSW} = 1.362\cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$

$Vu_{o.12D.16L.FSW} = 3.16\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.301\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.085\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.SWF} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.184\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.047\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.167\cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0\cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0\cdot kip$

$$Vu_{o.w.SWF} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.SWF} \qquad +-V_{ot\_w.SWF}$$

$Vu_{o.w.SWF} = 1.924\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.SWF} := F_{7.10}\cdot\max\left(V_{ot\_s.SWF} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$

$Vu_{o.s.SWF} = 1.362\cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.SWF} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$

$Vu_{o.12D.16L.SWF} = 3.16\cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5\cdot in$ $w_{pl.7.9} = 5\cdot in$ $x_{Le.7.9} = 1\cdot in$ $x_{we.7.9} = 4\cdot in$ $t_{pl.7.9} = 0.313\cdot in$

$h_{ef.7.9} = 3.719\cdot in$ $h_{a.7.9} = 8\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0\cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5\cdot in$ $A_{SE.7.9} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9}\cdot A_{SE.7.9}\cdot f_{uta}$ $N_{sa.7.9} = 70\cdot kip$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.034 \quad \text{or} \quad \left[\frac{Vu_{o.w.FSW}}{\left(0.75 \cdot V_{cbg\_op.7.9}\right)} + \frac{Vu_{i.w.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.576 \quad < 1.0$$

$$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.009 \quad \text{or} \quad \left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.441 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.929 \quad < 1.0$$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.07 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.579 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.02 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.473 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.918 < 1.0$$

$Result_{SWF}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWF}$ = "not required" < 1.2

$chk_{6.SWF}$ = "OK"

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.1} = 0.571 \cdot in$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.2} = 0.404 \cdot in$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.571\cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.404\cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.204\cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min(w_{pl.7.9}, w_{pl.7.10})}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5\cdot in$$

$l_{w\_reqd.FSW} := \max(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6})$ $l_{w\_reqd.FSW} = 2.5\cdot in$

****

****

g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $wgt_r = 56.542\cdot psf$ Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $wgt_w = 39.333\cdot psf$ Wall Dead Load

$wgt_{flr}^T = (41.15 \quad 40.4 \quad 39.84 \quad 39.42 \quad 39.08 \quad 38.8 \quad 38.58 \quad 37.78 \quad 37.66)\cdot psf$ Floor Dead Load

$W_{lat} = 45.986\cdot psf$ Wind Load

$w_{llf.0} = 250\cdot psf$ $w_{llf.0.ref} = $ "Floor Live Load Option 0"

$w_{llf.1} = 200\cdot psf$ $w_{llf.1.ref} = $ "Floor Live Load Option 1"

$w_{llf.2} = 150\cdot psf$ $w_{llf.2.ref} = $ "Floor Live Load Option 2"

$A_r := L_r\cdot W_r$ $A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\,ft^2$ Area of Roof/Floor

$A_f := L_f\cdot W_f$ $A_f^T = (123.7 \quad 145.7 \quad 167.6 \quad 189.5 \quad 211.4 \quad 233.3 \quad 255.2 \quad 277.2 \quad 299.1)\,ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$ Allowable Resisting Moment

$M_R^T = (530.9 \quad 543.2 \quad 555.6 \quad 567.9 \quad 580.2 \quad 592.5 \quad 604.8 \quad 617.1 \quad 629.4)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$ Overturning Moment due to Wind

$M_{OT.wind}^T = (48.6 \quad 56.7 \quad 64.8 \quad 72.9 \quad 81 \quad 89.1 \quad 97.2 \quad 105.3 \quad 113.4)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^T = (72.9 \quad 85 \quad 97.2 \quad 109.3 \quad 121.5 \quad 133.6 \quad 145.8 \quad 157.9 \quad 170.1)\cdot kip\cdot ft$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\left[\begin{array}{l} L_r\cdot W_r\cdot\left(wgt_r\right)\cdot\max\left(H_E\right) + L_r\cdot W_r\cdot\left(0.25w_{llr}\right)\cdot\max\left(H_E\right) \ldots \\ +\left(2L_f\cdot\max\left(H_E\right) + 2W_{ew}\cdot\max\left(H_E\right)\right)\cdot\dfrac{wgt_w\cdot\max\left(H_E\right)}{2} \ldots \\ + L_f\cdot W_f\cdot wgt_{flr}\cdot\dfrac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot\left(0.25w_{llf.0}\right)\cdot t_{floor} \end{array}\right]$$

Overturning Moment due to seismic

$$M_{OT.seismic}^{T} = (39.135 \quad 43.546 \quad 47.957 \quad 52.367 \quad 56.778 \quad 61.189 \quad 65.6 \quad 70.011 \quad 74.422)\cdot kip\cdot ft$$

$$1.5\,M_{OT.seismic}^{T} = (58.702 \quad 65.318 \quad 71.935 \quad 78.551 \quad 85.168 \quad 91.784 \quad 98.4 \quad 105.017 \quad 111.633)\cdot kip\cdot ft$$

$$M_{ot.w.net} := \left(1.5M_{OT.wind} - M_R\right) \qquad M_{ot.s.net} := \left(1.5M_{OT.seismic} - M_R\right)$$

$$M_{ot.w.net}^{T} = (-458 \quad -458.2 \quad -458.4 \quad -458.5 \quad -458.7 \quad -458.8 \quad -459 \quad -459.1 \quad -459.3)\cdot kip\cdot ft$$

$$M_{ot.s.net}^{T} = (-472.2 \quad -477.9 \quad -483.6 \quad -489.3 \quad -495 \quad -500.7 \quad -506.4 \quad -512.1 \quad -517.7)\cdot kip\cdot ft$$

$chk_{S7.15} := \text{if}\left(\max\left(M_{ot.w.net}, M_{ot.s.net}\right) \geq 0, \text{"NG"}, \text{"OK"}\right)$ $chk_{S7.15} = \text{"OK"}$

Sliding

$$WT := \begin{pmatrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min\left(H_E\right)\cdot wgt_w \ldots \\ +2W_{ew}\cdot\min\left(H_E\right)\cdot wgt_w + 2L_f\cdot W_f\cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^{T} = (169.422 \quad 171.532 \quad 173.641 \quad 175.75 \quad 177.859 \quad 179.969 \quad 182.078 \quad 184.187 \quad 186.296)\cdot kip$$

$$WT_{max} := \max\left[\begin{array}{l} A_r\cdot\left(wgt_r + w_{misc.r}\right) + A_f\cdot\left(wgt_{flr} + w_{misc.f}\right) \ldots \\ +2L_f\cdot\max\left(H_E\right)\cdot\left(wgt_w + w_{misc.w}\right) \ldots \\ +2\cdot W_{ew}\cdot\max\left(H_E\right)\cdot wgt_w + \left[A_r\cdot w_{llr} + A_r\cdot\left(w_{llf.0}\right)\right] \end{array}\right]$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 253.522\cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in\cdot 4in$ $A_{pad} = 16\cdot in^2$

$S_{bp} := 3ft$ Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad} = 7.165\cdot kip$ Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 447.8\cdot psi$ Compressive Stress per Pad

$F_{pad} := 800psi$ Allowable Stress per pad

$F_P := 0.7\cdot 0.85f'_{c.fdn}$ $F_P = 1785.1\cdot psi$ Allowable Stress of Fdn Concrete

$chk_{S7.16} := \text{if}\left(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, \text{"OK"}, \text{"NG"}\right)$ $chk_{S7.16} = \text{"OK"}$

$$wt_{pad.big} := \max\left[\frac{\max\left(WT_{max}\right)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad.big} = 9.751\cdot kip$ Weight per Bearing Pad

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Occupancy: S2
Allowable No. of Floors: One (1)
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$ $f_{pad.big} = 609.4 \cdot psi$ Compressive Stress per Pad

$chk_{S7.17} := if\left(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, "OK", "NG"\right)$ $chk_{S7.17} = "OK"$

$\mu := 0.25$ Coefficient of Friction

Wind $V_{net.w.sl} := max\left[0kip, \left[\mu \cdot \left[-(WT) + 1.5\left|P_{wind_3}\right| \cdot \left(A_r\right)\right] \ldots \right.\right.$
$\left.\left. + 1.5 W_{lat} L \cdot max\left(H_E\right)\right]\right]$ $V_{net.w.sl} = 0 \cdot kip$ Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := max\left[0kip, 1.5 \cdot \left[0.7 C_s \cdot \left(\begin{array}{l} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{array}\right) + -\mu \cdot \left(\begin{array}{l} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{array}\right)\right]\right]$

$V_{net.s.sl} = 0 \cdot kip$ Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$chk_{S7.18} := if\left(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, "Slide Plates Req'd", "OK"\right)$ $chk_{S7.18} = "OK"$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \left\| \begin{array}{l} max\left(2, ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) \text{ if } \frac{ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ max\left[2, \left(ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] \text{ otherwise} \end{array} \right.$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (10 \quad 12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (10 \quad 12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26)\,ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3\,ft$ on center.

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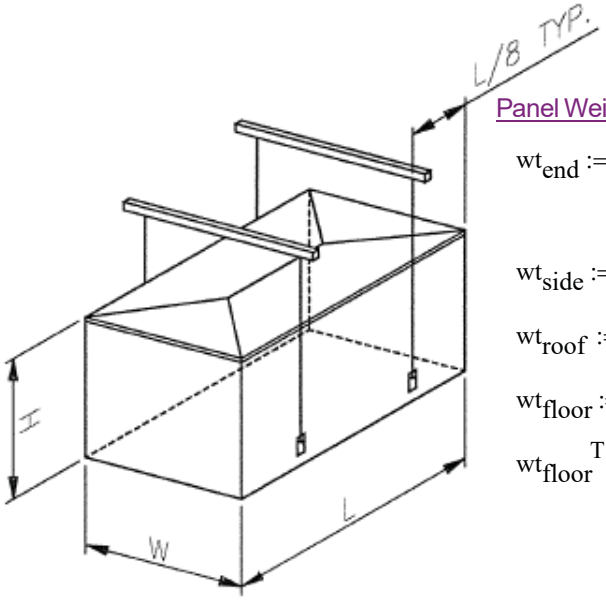
8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5153 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 442 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 679 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (480 \quad 471 \quad 465 \quad 460 \quad 456 \quad 453 \quad 450 \quad 441 \quad 439) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) M/(0.9f_yd)			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	12.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.016	0.042
12.00	14.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.029	0.016
12.00	16.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.044	0.011
12.00	18.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.062	0.038
12.00	20.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.082	0.065
12.00	22.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.105	0.092
12.00	24.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.131	0.119
12.00	26.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.159	0.146
12.00	28.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.189	0.172
12.00	9.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.000	0.009
12.00	10.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.005	0.015
12.00	10.67	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.008	0.020
12.00	18.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.058	0.029
12.00	20.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.078	0.050
12.00	22.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.100	0.071
12.00	24.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.124	0.092
12.00	26.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.150	0.113
12.00	28.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.179	0.133
12.00	12.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.014	0.029
12.00	14.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.026	0.011
12.00	16.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.040	0.008
12.00	18.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.056	0.026
12.00	20.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.075	0.044
12.00	22.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.096	0.062
12.00	24.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.120	0.080
12.00	26.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.145	0.099
12.00	28.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.173	0.117

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b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

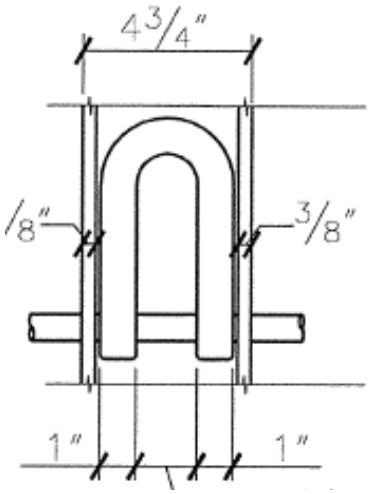
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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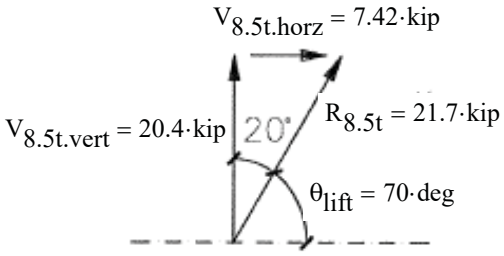


$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

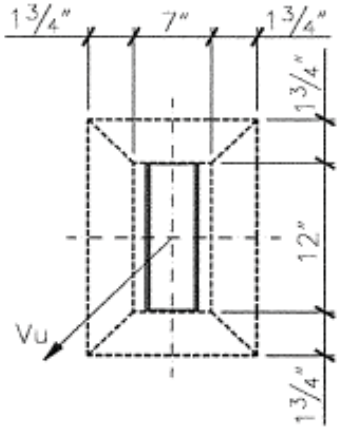
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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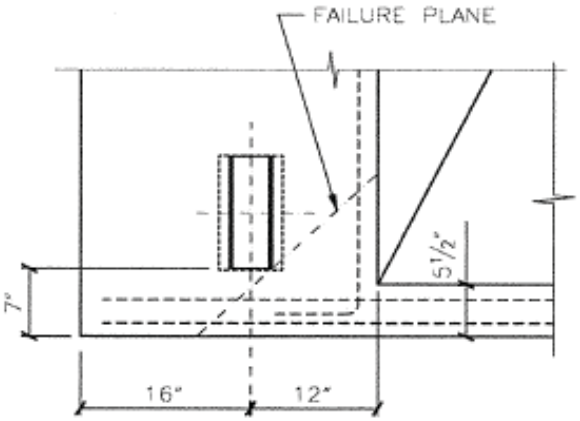
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

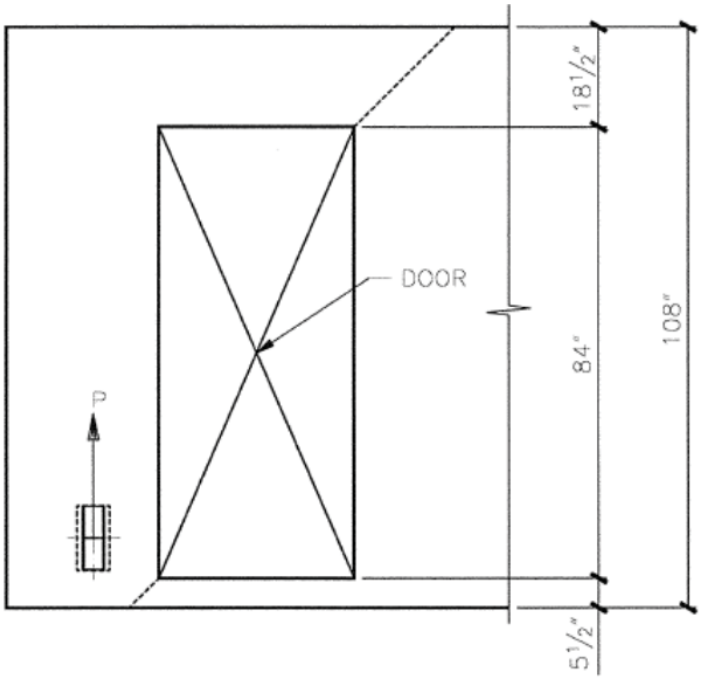
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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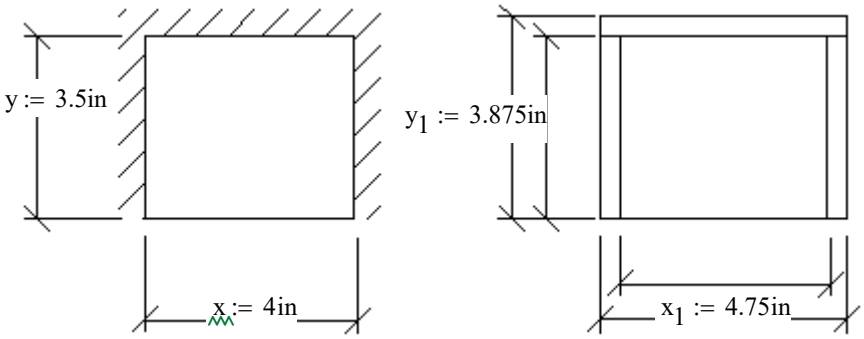
Bearing Pressure

Vertical



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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

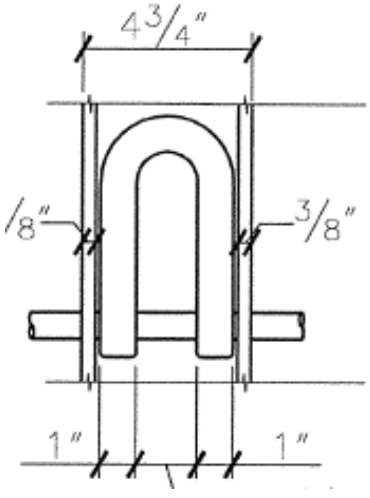
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

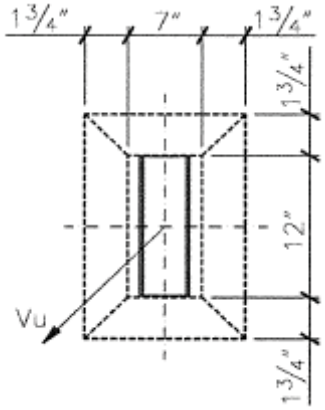
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

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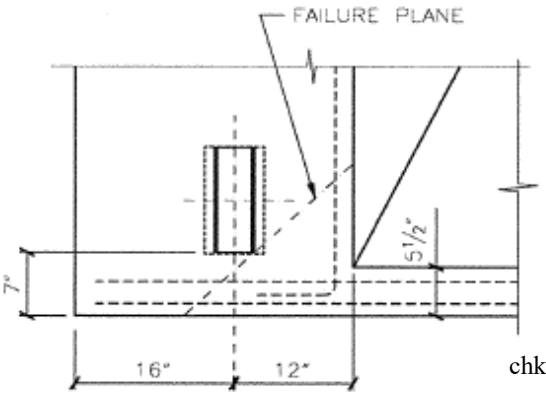
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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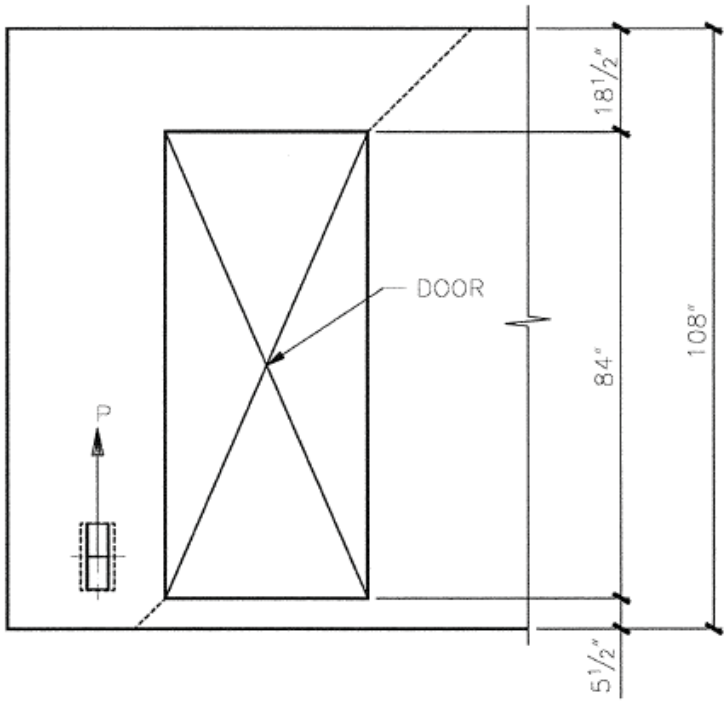
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

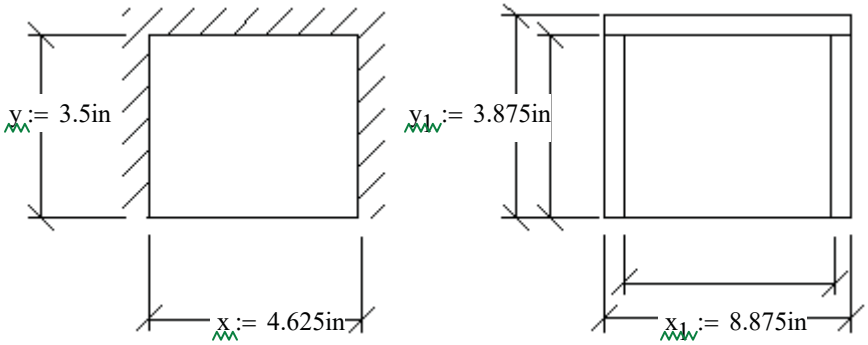
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

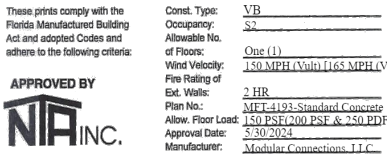
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



MODULAR CONNECTIONS, LLC.
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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 19 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 13.4 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2\max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 24.7 \cdot kip$ Wall Dead Load

$w_{ew} := 2\max(wt_{end})$ $w_{ew} = 10.3 \cdot kip$ Weight of End Wall

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Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 67.5 \cdot kip$ Factored Dead Load of Shelter

$$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}} \quad CG = 6.9\,ft$$ Center of gravity from bottom of shelter

Tie Down Bracket

$Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 122.8 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}} \quad F_{turn} = 28.7 \cdot kip$$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 206.5 \cdot kip \cdot ft$ Overtruning Moment

$$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2} \quad M_{OT.Resist} = 258.1 \cdot kip \cdot ft$$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -51.6 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}} \quad F_{OT.trans} = -6.1 \cdot kip$$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans} DL$ $F_{net.slide.trans} = 15.2 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 7.6 \cdot kip$ Maximum Shear Force Per Tie-Down, θ_{min}

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$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 7.6 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$

$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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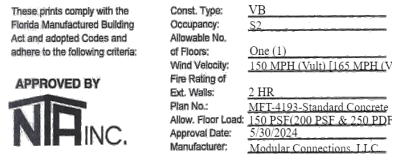
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right) \qquad \phi P_{pl.FlrTie} = 26.928 \cdot kip$$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.283$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.663$ < 1.0

$$\max\left(\frac{T_{tie.max}}{\phi P_{pl.FlrTie}}\right) + \frac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.946$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 0.946$ < 1.2

$chk_{8.Flr.Tie}$ = "OK"

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9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

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Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 5 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 2.25 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 21.67 \quad U_{R3.25} = 0.046$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2)\cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 5 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 2.25 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 26.67 \quad U_{R4.75} = 0.037$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (11.292 \quad 13.292 \quad 15.292 \quad 17.292 \quad 19.292 \quad 21.292 \quad 23.292 \quad 25.292 \quad 27.292)\,ft$ Length of Floor

$wgt_{flr}^T = (41.147 \quad 40.396 \quad 39.841 \quad 39.415 \quad 39.077 \quad 38.803 \quad 38.575 \quad 37.781 \quad 37.662)\cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 4.184\cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 3.816\cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.883 \\ 0.68 \\ 14.69 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 17.253 \quad U_F = 0.058$

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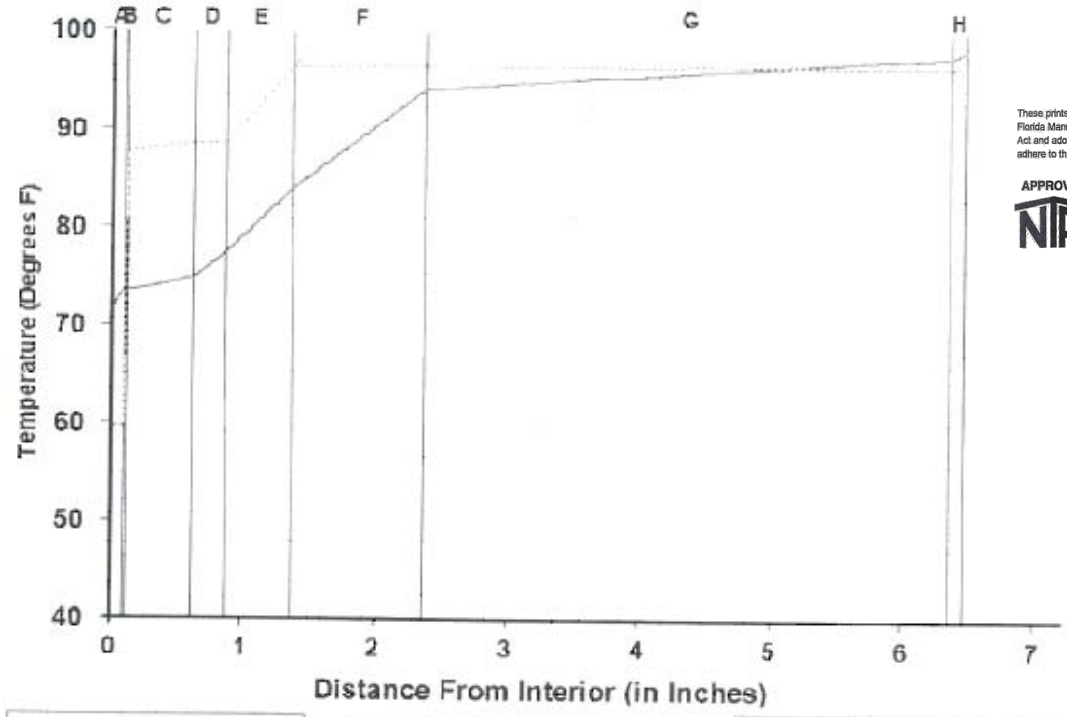
Dewpoint Analysis - Dow Chemical

Sam Cain



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Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

May 17 2006 12:16:28 PM

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1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 65.753 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 935.2 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 1987 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$ Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.0101$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.242 \cdot in^2$ Req'd Area of Steel per foot Width

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 351 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG")$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 7.516\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 15.032\,ft$

Maximum allowable opening width

$w_{d.max} := \max(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4})$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

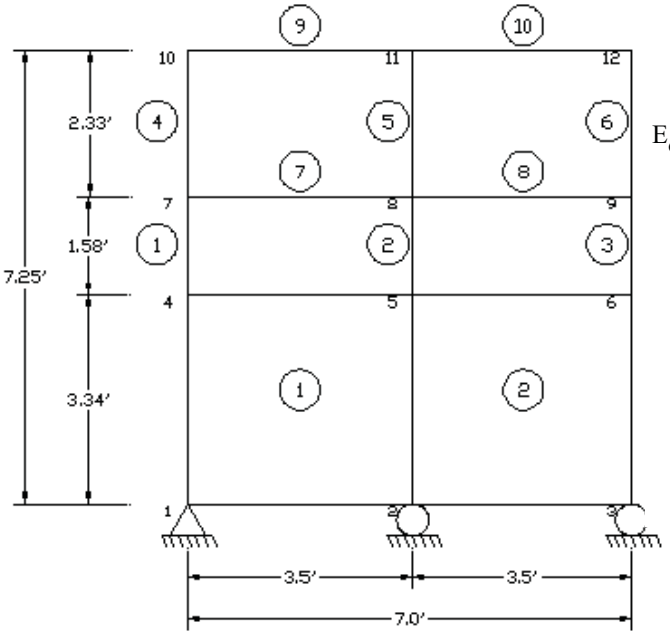
$chk_{opn.width} := if(w_d \geq w_{d.max}, "OK", "NG")$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.436 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.436 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.436 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2})$ In-Plane Force at Endwall



$V_u = 4.436 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in^2)	I (in^4)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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1090 Industrial Blvd.
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$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.436 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.309 \cdot kip \cdot ft$

Shear

$\phi Vc_{6d.opn} := \phi_V \cdot 0.85 \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot \left(d_7 - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00113$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot \max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.142 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$ Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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Email: info@modularconnections.com

APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPENDIX "B"

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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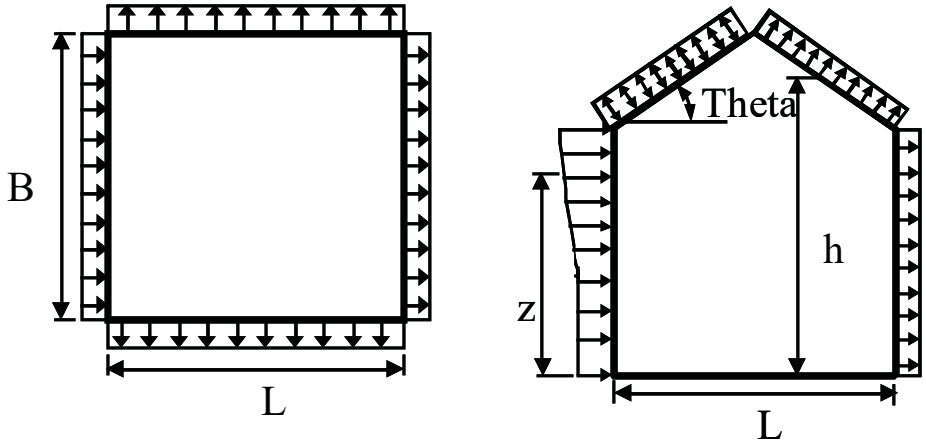
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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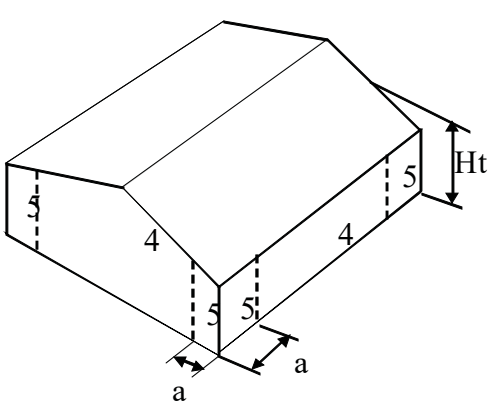
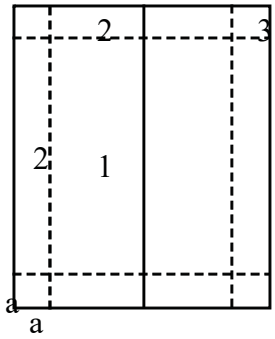
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

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APPENDIX "C"

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

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MODULAR CONNECTIONS, LLC

" BUILDING WIDTH ----------12 ft "

" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"

" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "

" BUILDING CODE "

" IBC 2021 (ASCE 7-16) "

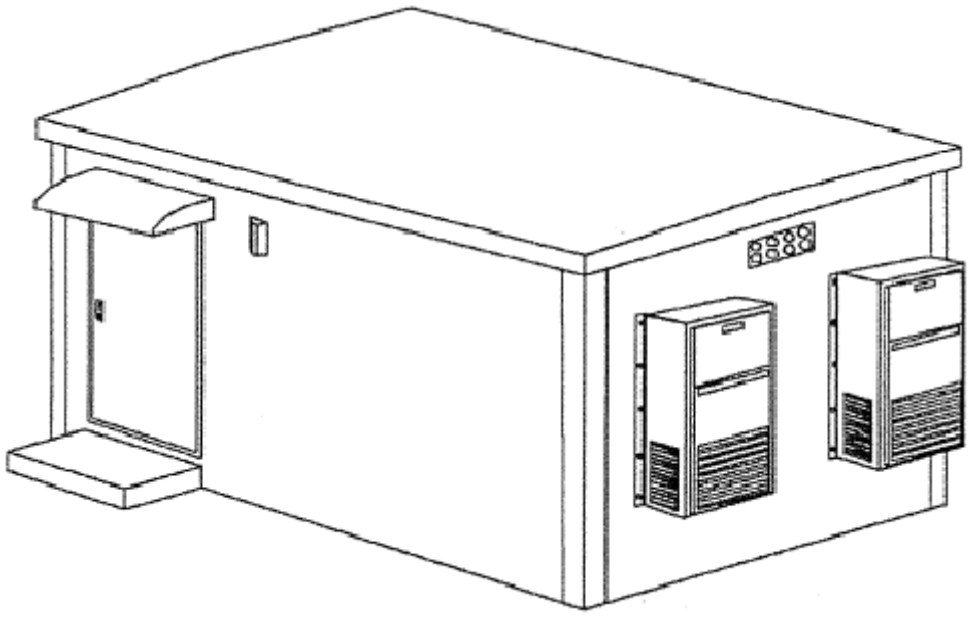
" & APPLICABLE REFERENCES "

" STANDARD CALCULATION PACKAGE - SINGLE WIDE BUILDING, 12 ft WIDE, SECTION #6"

" REVISION - 0: DATE MARCH 11, 2024 "

""

Cover =



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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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" APPENDIX A - BULLET RESISTANCE TEST "
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I CERTIFY THE ATTACHED ANALYSIS WAS PREPARED UNDER MY DIRECT SUPERVISION AND UNDER THE LAWS OF THIS STATE, OF WHICH I AM A REGISTERED DESIGN PROFESSIONAL.

LONNIE A MARVEL
LICENSE
No. 83750
STATE OF FLORIDA
PROFESSIONAL ENGINEER

2024.05.29 15:35:32-04'00'

MODULAR CONNECTIONS, LLC.
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Bessemer, AL 35022
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Email: info@modularconnections.com

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PROJECT STATEMENT

Prepare calculations for submission to building officials of pre-fabricated pre-cast concrete building manufactured by Modular Connections, LLC. Design shall include details, notes, specifications, and drawings.

BOD =

"Building Code ---------- IBC 2021 (ASCE 7-16)"
"Risk Category ---------- II"
"Building Width ---------- 12 ft"
" BUILDING LENGTH ---------- 12 ft - 28 ft , 2 ft Increments"
" MAXIMUM WALL PANEL HEIGHT ---------- 9 ft , 10 ft , & 10.67 ft "
"Average Floor Dead Load ---------- 39.2 psf "
"Floor Live Load Option 1 ---------- 250 psf "
"Floor Live Load Option 2 ---------- 200 psf "
"Floor Live Load Option 3 ---------- 150 psf "
"Roof Dead Load ---------- 46.7 psf"
"Roof Live Load ---------- 35 psf"
"Ground Snow Load ---------- 31.5 psf"
"Snow Load Importance Factor ---------- 1"
"Snow Exposure Factor ---------- 1.2"
"Snow Thermal Factor ---------- 1.2"
"Flat Roof Snow Load ---------- 31.8 psf"
"Wind Velocity ---------- 165 mph"
"Exposure Category ---------- C"
"Internal Pressure Coefficient ---------- 0.18"
"Horiztonal Wind Pressure, MWFRS ---------- 43.3 psf"
"Uplift Wind Pressure, MWFRS ---------- 64.7 psf"
"Zone 4 Wind Pressure, C & C ---------- 64.5 psf"
"Zone 5 Wind Pressure, C & C ---------- 79.6 psf"
"Zone 1 Wind Pressure, C & C ---------- 94.7 psf"
"Zone 2 Wind Pressure, C & C ---------- 124.9 psf"
"Zone 3 Wind Pressure, C & C ---------- 170.2 psf"
"(ASD) Zone 4 Wind Pressure, C & C ---------- 38.7 psf"
"(ASD) Zone 5 Wind Pressure, C & C ---------- 47.7 psf"
"Site Class ---------- D"
"Seismic Design Category ---------- D"
"Seismic F. Resisting Syst. ---------- Bearing Wall - Int. Precast Conc. Shear Walls"
"Analysis Procedure ---------- Equivalent Lateral-Force Procedure"
"Seismic Importance Factor ---------- 1"
"Spectral Accel., SDS, Short Period ---------- 0.96"
"Spectral Accel., SD1, 1-s Period ---------- 0.84"
"Seismic Response Coefficient, Cs ---------- 0.24"

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

1. DESIGN CRITERIA

a. Concrete

(Bldg) Specified 28 Day Concrete Compressive Strength $f'_{c.spec} = 5000 \cdot psi$ $\gamma_{LW} = 118 \cdot pcf$

$$E_{c.spec} := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_{c.spec} \cdot psi} \qquad E_{c.spec} = 2991 \cdot ksi$$

Conservatively for Design Utilize 28 Day Concrete Compressive Strength Sand LW $f'_c = 4500 \cdot psi$

$$E_c := \left(\frac{\gamma_{LW}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi} \qquad E_c = 2838 \cdot ksi$$

(Bldg) Sand lightweight concrete mix obtaining a $f_c := 2500 \cdot psi$ minimum compressive strength in < 28 days for shipping and handling

(foundation) Normal weight concrete mix obtaining $f'_{c.fdn} = 3000 \cdot psi$ minimum compressive strength in 28 days

b. Concrete Reinforcement

ASTM A615 grade 60 & ASTM A185 smooth welded wire fabric $f_y = 60 \cdot ksi$

c. Connections Properties

Plates, bars, etc... : ASTM A36 ($F_y = 36 \cdot ksi$) Headed studs : ASTM A108 ($F_y = 36 \cdot ksi$)

Welding electrodes : E70XX

d. Loads

Floor Live Load

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$w_{misc.f} = 4 \cdot psf$ $w_{misc.f.ref}$ = "Misc. Floor Dead"

Roof Live Load

$w_{llr} = 35 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$w_{misc.r} = 7 \cdot psf$ $w_{misc.r.ref}$ = "Misc. Roof Dead Load"

Snow Load

$$SL\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"pf=0.7CeCtIspg"} & \text{"Design Snow Load, Flat Roofs, Eqn (7.3-1)"} \\ 1.2 & \text{"Ce, Exp. Fact., Tbl (7.3-1)- Shelt, Surf Rough Cat. B"} \\ 1.2 & \text{"Ct, Therm Fact, Tbl (7.3-2)- Unheat \& Open Air"} \\ 1 & \text{"Is, Snow Load Importance Fact.,Tbl (1.5-2)"} \\ 31.5 & \text{"pg, Ground Snow Load"} \\ 31.5 & \text{"pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"} \end{pmatrix}$$

$p_{f.calc} := 0.7 \cdot C_e \cdot C_t \cdot I_S \cdot p_g$ $p_{f.calc} = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$p_m = 31.5 \cdot psf$ $p_{m.ref}$ = "pm, Min Snow Load, pm=pg*Is, Sec (7.3.4)"

$p_f := \max(p_{f.calc}, p_m)$ $p_f = 31.8 \cdot psf$ $p_{f.ref}$:= "Flat Roof Snow Load"

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer: Modular Connections, LLC

Wind Load

Main Wind Force Resisting System (MWFRS)

$$\text{MWFRS\_Param} = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV\^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 0.85 & \text{"Kd, Wind Direct. Fact., Tbl(26.6-1)"} \\ 0.85 & \text{"Kz, Vel. Pres. Exp. Coeff., Tbl (26.10-1) z}\leq\text{15ft, Exp C"} \\ 1 & \text{"Ke, Ground Elev. Fact., Tbl (26.9-1) 0 Elev."} \\ 1 & \text{"Kzt, Topo. Fact., Sec (26.8.2), No Topo. Considered"} \\ 0.85 & \text{"G, Gust Effect Fact., Sec (26.11.1)"} \\ 0.8 & \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ -0.5 & \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ -0.7 & \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ -1.3 & \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \\ 165 & \text{"Basic Wind Speed (mph)"} \\ 0.18 & \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"} \\ \text{"P"} & \text{"PA, Design MWFRS Wind Pressure, Eqn (27.3-1)"} \end{pmatrix}$$

$$q_h := 0.00256 \cdot K_d \cdot K_z \cdot K_e \cdot K_{zt} \cdot \left(\frac{V_w}{\text{mph}}\right)^2 \cdot \text{psf} \qquad q_h = 50.355 \cdot \text{psf} \qquad q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$$

$$C_p = \begin{pmatrix} 0.8 & 0.8 \\ -0.5 & -0.5 \\ -0.7 & -0.7 \\ -1.3 & -1.3 \end{pmatrix} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$GC_{pi} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \qquad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

(Positive internal pressure)

$$P_{A\_pi} := q_h \cdot \left(G \cdot C_p - GC_{pi}\right)$$

$$P_{A\_pi} = \begin{pmatrix} 25.18 & 43.31 \\ -30.46 & -12.34 \\ -39.03 & -20.9 \\ -64.71 & -46.58 \end{pmatrix} \cdot \text{psf} \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

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Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

(Negative internal pressure)

$$P_{A\_ni} := q_h \cdot \left[G \cdot C_p - \left(-GC_{pi} \right) \right]$$

$$P_{A\_ni} = \begin{pmatrix} 43.31 & 25.18 \\ -12.34 & -30.46 \\ -20.9 & -39.03 \\ -46.58 & -64.71 \end{pmatrix} \cdot psf \qquad C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$P_{wind} = \begin{pmatrix} 43.31 \\ 30.46 \\ 39.03 \\ -64.71 \end{pmatrix} \cdot psf$$

Max Values For Area

$$C_{p.ref} = \begin{pmatrix} \text{"MWFRS, Ext Pres Coeff for Windward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Leeward Walls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Sidewalls, Fig (27.3-1)"} \\ \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"} \end{pmatrix}$$

$$W_{lat} := \max\left(\begin{pmatrix} P_{A\_pi_{0,0}} - P_{A\_pi_{1,0}} & P_{A\_pi_{0,1}} - P_{A\_pi_{1,1}} \\ P_{A\_ni_{0,0}} - P_{A\_ni_{1,0}} & P_{A\_ni_{0,1}} - P_{A\_ni_{1,1}} \end{pmatrix} \right)$$

$W_{lat} = 55.64 \cdot psf$

Maximum Overturning / Sliding Wind Pressure

Component & Cladding (CC)

$$CC\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ \text{"C"} & \text{"Exposure Category, Sec (26.7.3)"} \\ \text{"qh=0.000256KzKztKdKeV^2"} & \text{"V, Velocity Pres., Eqn (26.10-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ -1.1 & \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ 1 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ -1.4 & \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ -1.7 & \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ -2.3 & \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ 0.3 & \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ -3.2 & \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \\ 165 & \text{"Basic Wind Speed (mph)"} \\ \text{"Zones 4 \& 5"} & \text{"(-1.1 * 0.9945 Ajust Fact. 20SF Wall Area, Worst Case)"} \\ \text{"Pc"} & \text{"Design CC Wind Pressure, Eqn (30.3-1)"} \end{pmatrix}$$

$q_h = 50.355 \cdot psf$

$q_{h.ref} = \text{"V, Velocity Pres., Eqn (26.10-1)"}$

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$$GC_{p.cc} = \begin{pmatrix} 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.1 & -1.1 \\ 1 & 1 \\ -1.4 & -1.4 \\ 0.3 & 0.3 \\ -1.7 & -1.7 \\ 0.3 & 0.3 \\ -2.3 & -2.3 \\ 0.3 & 0.3 \\ -3.2 & -3.2 \end{pmatrix} \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$$GC_{pi.cc} = \begin{pmatrix} 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \\ 0.18 & -0.18 \end{pmatrix} \quad GC_{pi.ref} = \text{"MWFRS Int. Pres. Coeff., Tbl (26.11-1)"}$$

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(Positive internal pressure)

$$P_{c\_pi} := q_h \cdot \left(GC_{p.cc} - GC_{pi.cc}\right)$$

$$P_{c\_pi} = \begin{pmatrix} 41.3 & 59.4 \\ -64.5 & -46.3 \\ 41.3 & 59.4 \\ -64.5 & -46.3 \\ 41.3 & 59.4 \\ -79.6 & -61.4 \\ 6 & 24.2 \\ -94.7 & -76.5 \\ 6 & 24.2 \\ -124.9 & -106.8 \\ 6 & 24.2 \\ -170.2 & -152.1 \end{pmatrix} \cdot p \; GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

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(Negative internal pressure)

$$P_{c\_ni} := q_h \cdot \left[GC_{p.cc} - \left(-GC_{pi.cc} \right) \right]$$

$$P_{c\_ni} = \begin{pmatrix} 59.4 & 41.3 \\ -46.3 & -64.5 \\ 59.4 & 41.3 \\ -46.3 & -64.5 \\ 59.4 & 41.3 \\ -61.4 & -79.6 \\ 24.2 & 6 \\ -76.5 & -94.7 \\ 24.2 & 6 \\ -106.8 & -124.9 \\ 24.2 & 6 \\ -152.1 & -170.2 \end{pmatrix} \cdot p \quad GC_{p.cc.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 3, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

Max Values
For Area

$$P_{wind.c} = \begin{pmatrix} 64.5 \\ 64.5 \\ 79.6 \\ 94.7 \\ 124.9 \\ 170.2 \end{pmatrix} \cdot psf \quad GC_{p.cc.max.ref} = \begin{pmatrix} \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"} \\ \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"} \\ \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"} \end{pmatrix}$$

$P_{W.c} := P_{wind.c_0}$ $\quad P_{W.c} = 64.5 \cdot psf$ $\quad GC_{p.cc.max.ref_0} = \text{" CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{L.c} := P_{wind.c_1}$ $\quad P_{L.c} = 64.5 \cdot psf$ $\quad GC_{p.cc.max.ref_1} = \text{" CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"}$

$P_{LC.c} := P_{wind.c_2}$ $\quad P_{LC.c} = 79.6 \cdot psf$ $\quad GC_{p.cc.max.ref_2} = \text{"CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"}$

$P_{R.c} := P_{wind.c_3}$ $\quad P_{R.c} = 94.7 \cdot psf$ $\quad GC_{p.cc.max.ref_3} = \text{"CC, MIN Ext Pres Coeff, Zone 1 \& 1', Fig (30.3-2A)"}$

$P_{Re.c} := P_{wind.c_4}$ $\quad P_{Re.c} = 124.9 \cdot psf$ $\quad GC_{p.cc.max.ref_4} = \text{"CC, MIN Ext Pres Coeff, Zone 2, Fig (30.3-2A)"}$

$P_{Rc.c} := P_{wind.c_5}$ $\quad P_{Rc.c} = 170.2 \cdot psf$ $\quad GC_{p.cc.max.ref_5} = \text{"CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"}$

$$P_{Up.c} := \text{mean}\left(P_{R.c}, P_{Re.c}, P_{Rc.c}\right) \qquad P_{Up.c} = 129.917 \cdot psf$$

Max Roof Uplift Pressure (average)

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Seismic Load

$$EQ\_Param = \begin{pmatrix} \text{"IBC 2021 (ASCE 7-16)"} & \text{"Design Code"} \\ \text{"II"} & \text{"Risk Category. Tbl (1.5-1)"} \\ 1 & \text{"Seismic Importance Fact., Tbl (1.5-2)"} \\ \text{"V = CsW"} & \text{"Seismic Base Shear, Eqn (12.8-1)"} \\ \text{"D"} & \text{"Site Class, Sec (11.4.3)"} \\ 1.44 & \text{"Spectral Accel. for Short Per., Figs (22-1,3,5,6,7,8)"} \\ 0.74 & \text{"Spectral Accel. for 1s Per., Figs (22-2,4,5,6,7,8)"} \\ 1 & \text{"Site Coeff. for Short Per., Tbl (11.4-1)"} \\ 1.7 & \text{"Site Coeff. for Long Per., Tbl (11.4-2)"} \\ 4 & \text{"Seismic Response Mod. Fact., Tbl (12.2-1)"} \\ \text{"A.5 - Bearing Wall System,"} & \text{"Seismic Force Resisting System"} \\ \text{"Intermediate Precast Shear Walls"} & \text{"(SFRS)"} \\ 2.5 & \text{"Seismic Overstrength Mod. Fact., Tbl (12.2-1)"} \\ 4 & \text{"Seismic Deflec. Amp. Mod. Fact., Tbl (12.2-1)"} \\ \text{"S\_MS"} & \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"} \\ \text{"S\_DS"} & \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"} \\ \text{"Cs"} & \text{"Seismic Response Coeff., Eqn (12.8-2)"} \\ 0.02 & \text{"CtE, Approx Per. Parameter, Tbl (12.8-2)"} \\ 0.75 & \text{"x, Approx Per. Parameter, Tbl (12.8-2)"} \\ \text{"Ta"} & \text{"Approx Per., Eqn (12.8-7)"} \\ \text{"S\_M1"} & \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"} \\ \text{"S\_D1"} & \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"} \\ \text{"Cs max"} & \text{"Max Seismic Response Coeff., Eqn (12.8-3)"} \\ \text{"CS min"} & \text{"Min Seismic Response Coeff., Eqn (12.8-6)"} \end{pmatrix}$$

$S_{MS} := F_a \cdot S_s$ $S_{MS} = 1.44$ $S_{MS.ref} = \text{"Spectral Accel. Adj for Site Class,Short Per., Eqn (11.4-1)"}$

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.96$ $S_{DS.ref} = \text{"5% Damped Spectral Accel. ,Short Per., Eqn (11.4-3)"}$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 1.258$ $S_{M1.ref} = \text{"Spectral Accel. Adj for Site Class,1s Per., Eqn (11.4-2)"}$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.839$ $S_{D1.ref} = \text{"5% Damped Spectral Accel. ,1s Per., Eqn (11.4-4)"}$

$$H_E = \begin{pmatrix} 9.479 \\ 10.479 \\ 11.146 \end{pmatrix} \text{ft} \qquad H_{E.ref} = \text{"Exterior Height of Building"}$$

$$Ta := C_{t.E} \cdot \min\left[\left(\frac{H_E - t_{r.r}}{\text{ft}}\right)^x\right] \qquad Ta = 0.104 \quad Ta_{ref} = \text{"Approx Per., Eqn (12.8-7)"}$$

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$T_0 := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_0 = 0.175$ $T_s := \left(\frac{S_{D1}}{S_{DS}}\right)$ $T_s = 0.874$ $T_L := 12$

$C_{s.calc} := \frac{S_{DS} \cdot I_E}{R}$ $C_{s.calc} = 0.24$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$\frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 2.014$ $\frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 232.156$ $1.5 \cdot \frac{S_{D1}}{Ta \cdot \left(\frac{R}{I_E}\right)} = 3.021$ $1.5 \cdot \frac{S_{D1} \cdot T_L}{Ta^2 \cdot \left(\frac{R}{I_E}\right)} = 348.235$

$$C_{s.max.1} := \begin{cases} \frac{S_{D1}}{\min(Ta) \cdot \left(\frac{R}{I_E}\right)} & \text{if } \min(Ta) \le T_L \\ \frac{S_{D1} \cdot T_L}{\min(Ta)^2 \cdot \left(\frac{R}{I_E}\right)} & \text{otherwise} \end{cases}$$

$C_{s.max.1} = 2.014$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$C_{s.max} := \text{if}\left(\min(Ta) \le T_L \wedge S_1 \ge 0.2, 1.5 \cdot C_{s.max.1}, C_{s.max.1}\right)$ $C_{s.max} = 3.021$

$C_{s.max.ref} = \text{"Max Seismic Response Coeff., Eqn (12.8-3)"}$

$$Exemption := \begin{cases} \text{"Exempt 11.4.8"} & \text{if } SC = \text{"E"} \wedge S_1 \ge 0.2 \wedge Ta \le T_s \wedge S_s \ge 1.0 \\ \text{"Exempt 11.4.8"} & \text{if } SC = \text{"D"} \wedge S_1 \ge 0.2 \\ \text{"Not Exempt 11.4.8"} & \text{otherwise} \end{cases}$$

$Exemption = \text{"Exempt 11.4.8"}$

$0.044 \cdot S_{DS} \cdot I_E = 0.0422$ $\frac{0.5 \cdot S_1}{\frac{R}{I_E}} = 0.0925$

$C_{s.min} := \max\left(0.01, 0.044 \cdot S_{DS} \cdot I_E, \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\frac{R}{I_E}}, 0\right)\right)$ $C_{s.min} = 0.093$

$C_{s.min.ref} = \text{"Min Seismic Response Coeff., Eqn (12.8-6)"}$

$C_s := \max\left(\min\left(C_{s.calc}, C_{s.max}\right), C_{s.min}\right)$ $C_s = 0.24$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$SDC_1 = \text{"D"}$ $SDC1_{ref} = \text{"Table 11.6-1"}$

$SDC_2 = \text{"D"}$ $SDC2_{ref} = \text{"Table 11.6-2"}$

$SDC = \text{"D"}$ Seismic Design Category

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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2) ROOF

a.) Code Requirements:

- Concrete Strength $f'_c = 4500 \cdot psi$, $\gamma_c = 118 \cdot pcf$ Weight of Concrete, Reinforcement yield $f_y = 60 \cdot ksi$

Roof Edge Thicknesses $t_{r.e} = 4 \cdot in$ Thickness @ ridge $t_{r.r} = 5.5 \cdot in$ $t_{r.a} = 4.75 \cdot in$ Average Roof Thicknesses

$A_{s.mim.2} := t_{r.r} \cdot 12in \cdot \rho_{t\_s.min}$ $A_{s.mim.2} = 0.1188 \cdot in^2$ Min. Area of Rebar / 1ft width

- Utilizing roof edge concrete thickness for shear and mean roof thickness for flexure
- $\rho_{t\_s.min.ref}$ = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)" $\rho_{t\_s.min} = 0.0018$

b.) Loads / Load Combinations

$DL_{r.mn} := t_{r.a} \cdot \gamma_c$ $DL_{r.mn} = 46.71 \cdot psf$ $DL_{r.mn.ref} :=$ "Min. Roof DL" $w_{misc.r} = 7 \cdot psf$ MISC. DL

$DL_{r.mx} := DL_{r.mn} + w_{misc.r}$ $DL_{r.mx} = 53.71 \cdot psf$ $DL_{r.mx.ref} :=$ "Max. Roof DL"

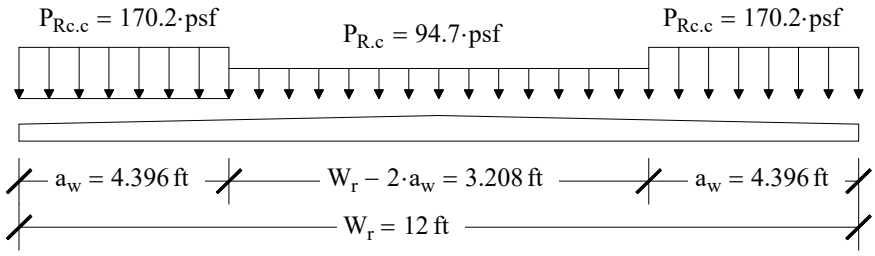
$w_{llr} = 35 \cdot psf$ $w_{llr.ref}$ = "Roof Live Load"

$p_f = 31.752 \cdot psf$ $p_{f.calc.ref}$ = "Design Snow Load, Flat Roofs, Eqn (7.3-1)"

$P_{Rc.c} = 170.2 \cdot psf$ $GC_{p.cc.max.ref_5}$ = "CC, MIN Ext Pres Coeff for Roof, Zone 3, Fig (30.3-2A)"

$P_{R.c} = 94.7 \cdot psf$ $GC_{p.cc.max.ref_3}$ = "CC, MIN Ext Pres Coeff, Zone 1 & 1', Fig (30.3-2A)"

$a_w := \max\left(0.1 \cdot \min(L, W_{ew}), 0.4 \cdot H_E - \frac{Rise}{2}, 3ft\right)$ $a_w = 4.396\,ft$ Width of Edge and Corner Wind Zone



$M_{r.wind.Up} := -P_{R.c} \cdot \frac{W_r^2}{8} - (P_{Rc.c} - P_{R.c}) \cdot \frac{a_w^2}{2}$ $M_{r.wind.Up} = -29.206 \cdot \frac{kip \cdot in}{ft}$ Moment Due to Wind Uplift

$w_{up.wind.eq} := \frac{8 \cdot M_{r.wind.Up}}{W_r^2}$ $w_{up.wind.eq} = -135.211 \cdot psf$ Equivalent Uniform Load, Wind Uplift

$E_{v.r.mx} := 0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mx} = 10.312 \cdot psf$ Maximum Downward Roof Seismic Load

$E_{v.r.mn} := -0.2 \cdot S_{DS} \cdot DL_{r.mx}$ $E_{v.r.mn} = -10.312 \cdot psf$ Minimum Upward Roof Seismic Load

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Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$LRFD = \begin{pmatrix} \text{"1a"} & \text{"1.4(D+F)"} \\ \text{"2a"} & \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"3a"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"3b+"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"3b-"} & \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"4a+"} & \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"4a-"} & \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"5a+"} & \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"5a-"} & \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"6a+"} & \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"6a-"} & \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"7a+"} & \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"7a-"} & \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Wu_{S2.0} = \begin{pmatrix} 75.19 \\ 81.95 \\ 120.45 \\ 120.45 \\ 52.84 \\ 81.95 \\ 14.34 \\ 81.11 \\ 60.49 \\ 42.04 \\ -93.17 \\ 52.35 \\ 31.73 \end{pmatrix} \cdot psf$$

Factored Area Loads

$b_{strip} = 12 \cdot in$ Strip width

$l_r = 11\ ft$ $l_{r.ref} = \text{"Max Span of Roof Panel"}$

$w_{u.mx} := \max(Wu_{S2.0})$ $w_{u.mx} = 120.45 \cdot psf$ Maximum Roof Factored Load, Roof Live Load 0

$w_{u.mn} := \min(Wu_{S2.0})$ $w_{u.mn} = -93.174 \cdot psf$ Minimum Roof Factored Load, Roof Live Load 0

$Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$ $Vu_{ref} = \text{"Ultimate Shear"}$

Roof Live Load

$Mu_{2.1.mx} := |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r^2}{8}$ $Vu_{2.1.mx} := |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r}{2}$

$Mu_{2.1.mx} = 21.862\ kip \cdot in$ $Vu_{2.1.mx} = 0.662\ kip$

Option 1

$Mu_{2.1.mn} := \min\left(0kip \cdot in, |w_{u.mn}| \cdot b_{strip} \cdot \frac{l_r^2}{8}\right)$ $Vu_{2.1.mn} := \min\left(0kip, |w_{u.mx}| \cdot b_{strip} \cdot \frac{l_r}{2}\right)$

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $Vu_{2.1.mn} = 0 \cdot kip$

c.) Member Design - Roof Live Load $w_{llr} = 35 \cdot psf$ $w_{llr.ref} = \text{"Roof Live Load"}$

$B_{2.1} = \text{"#4"}$ $B_{2.1.ref} = \text{"Transverse Rebar Size"}$ $S_{2.1} = 10 \cdot in$ $S_{2.1.ref} = \text{"Transverse Rebar Spacing"}$

$B_{2.2} = \text{"#4"}$ $B_{2.2.ref} = \text{"Longitudinal Rebar Size"}$ $S_{2.2} = 12 \cdot in$ $S_{2.2.ref} = \text{"Longitudinal Rebar Spacing"}$

$CC_{2.1a} = 1.5 \cdot in$ $CC_{2.1a.ref} = \text{"Clear Cover, From Bot. w/ Keyway"}$

$CC_{2.1b} = 2.25 \cdot in$ $CC_{2.1b.ref} = \text{"Clear Cover, From Bot. w/o Keyway"}$

$D_{2.1} = 0.5 \cdot in$ $D_{2.1.ref} = \text{"Roof Width Bar Diam."}$ $A_{2.1} = 0.2 \cdot in^2$ $A_{2.1.ref} = \text{"Roof Width Bar Area."}$

$As_{2.1} = 0.24 \cdot in^2$ $As_{2.1.ref} = \text{"Roof Width Area Reinf."}$ $D_{2.2} = 0.5 \cdot in$ $D_{2.2.ref} = \text{"Roof Length Bar Diam."}$

$A_{2.2} = 0.2 \cdot in^2$ $A_{2.2.ref} = \text{"Roof Length Bar Area."}$ $As_{2.2} = 0.2 \cdot in^2$ $As_{2.2.ref} = \text{"Roof Length Area Reinf."}$

$A_{s.mim.2} = 0.1188 \cdot in^2$ Minimum Area of Reinforcing per 1ft width

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$chk_{2.1} := if\left(As_{2.1} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.1} = "OK"$

$chk_{2.2} := if\left(As_{2.2} \geq A_{s.mim.2}, "OK", "NG"\right)$ $chk_{2.2} = "OK"$

Flexure (out of plane - Gravity)

$d_{F.2.1} := t_{r.a} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{F.2.1} = 3 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$As_{2.1} = 0.24 \cdot in^2$ Area/ft, Width Direction $\rho_{2.1} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.1}}$ $\rho_{2.1} = 0.0067$ $\rho_{ref} = "Reinf.\ Ratio"$

$a_{2.1} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $a_{2.1} = 0.314 \cdot in$ $a_{ref} = "Depth\ of\ Comp.\ Block,\ Eqn\ (22.2.2.4.1)"$

$M_{n.2.1} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.1} - \frac{a_{2.1}}{2}\right)$ $M_{n.2.1} = 40.9 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$c_{2.1} := \frac{a_{2.1}}{\beta_1}$ $c_{2.1} = 0.38 \cdot in$ $c_{ref} = "Depth\ of\ N/A\ Axis,\ Eqn\ (22.2.2.4.1)"$

$\varepsilon_{2.1} := 0.003 \cdot \frac{d_{F.2.1} - c_{2.1}}{c_{2.1}}$ $\varepsilon_{2.1} = 0.02067$ $\varepsilon_{ref} = "Tensile\ Strain,\ Extreme\ Layer"$

$\phi_{2.1} := if\left[\varepsilon_{2.1} \geq 0.005, 0.9, if\left[\varepsilon_{2.1} \leq 0.002, 0.65, 0.75 + \left(\varepsilon_{2.1} - 0.002\right) \cdot 50\right]\right]$

$\phi_{2.1} = 0.9$ $\phi_{ref} = "Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\phi M_{n.2.1} := \phi_{2.1} \cdot M_{n.2.1}$ $\phi M_{n.2.1} = 36.8 \cdot kip \cdot in$ $\phi M_{p.ref} = "Flexural\ Strength,\ +ve,\ Sect\ (22.3)"$

$Mu_{2.1.mx} = 21.9 \cdot kip \cdot in$ $Mu_{p.ref} = "Ultimate\ Moment,\ +ve"$

$chk_{2.3} := if\left(\phi M_{n.2.1} \geq Mu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.3} = "OK"$

Shear (out of plane - Gravity)

$t_{key} = 0.75 \cdot in$ $t_{key.ref} = "Keyway\ thickness"$

$d_{V.2.1} := t_{r.e} - t_{key} - CC_{2.1a} - \frac{D_{2.1}}{2}$ $d_{V.2.1} = 1.5 \cdot in$ $d_{ref} = "Depth\ of\ Reinf."$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.2.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.1}}{10 \cdot in}}}\right)$ $\lambda_{s.2.1} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{2.1.a} := 8 \cdot \lambda_{s.2.1} \cdot \lambda \cdot \rho_{2.1}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.a} = 3.091 \cdot kip$

$Vc_{2.1.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.1}\right)$ $Vc_{2.1.b} = 10.264 \cdot kip$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$Vc_{2.1} := \min\left(Vc_{2.1.b}, Vc_{2.1.a}\right)$ $Vc_{2.1} = 3.091 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$\phi Vc_{2.1} := \phi_V \cdot Vc_{2.1}$ $\phi Vc_{2.1} = 2.318 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$Vu_{2.1.mx} = 0.662 \cdot kip$ $Vu_{ref} = "Ultimate\ Shear"$

$chk_{2.4} := if\left(\phi Vc_{2.1} \geq Vu_{2.1.mx}, "OK", "NG"\right)$ $chk_{2.4} = "OK"$

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Flexure (out of plane - Uplift)

$d_{F.2.3} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{F.2.3} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\rho_{2.3} := \frac{As_{2.1}}{b_{strip} \cdot d_{F.2.3}}$ $\quad \rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$a_{2.5} := \frac{As_{2.1} \cdot f_y}{0.85 \cdot f'_c \cdot b_{strip}}$ $\quad a_{2.5} = 0.314 \cdot in$ $\quad a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$M_{n.2.3} := As_{2.1} \cdot f_y \cdot \left(d_{F.2.3} - \frac{a_{2.5}}{2} \right)$ $\quad M_{n.2.3} = 22.9 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$c_{2.3} := \frac{a_{2.5}}{\beta_1}$ $\quad c_{2.3} = 0.38 \cdot in$ $\quad c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$\varepsilon_{2.3} := 0.003 \cdot \frac{d_{F.2.3} - c_{2.3}}{c_{2.3}}$ $\quad \varepsilon_{2.3} = 0.01081$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{2.3} := \text{if}\left[\varepsilon_{2.3} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{2.3} \leq 0.002, 0.65, 0.75 + (\varepsilon_{2.3} - 0.002) \cdot 50 \right] \right]$

$\phi_{2.3} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi M_{n.2.3} := \phi_{2.3} \cdot M_{n.2.3}$ $\quad \phi M_{n.2.3} = 20.6 \cdot kip \cdot in$ $\quad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$

$Mu_{2.1.mn} = 0 \cdot kip \cdot in$ $\quad Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$chk_{2.5} := \text{if}\left(\phi M_{n.2.3} \geq Mu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.5} = \text{"OK"}$

Shear (out of plane - Uplift)

$d_{V.2.4} := CC_{2.1a} + \frac{D_{2.1}}{2}$ $\quad d_{V.2.4} = 1.75 \cdot in$ $\quad d_{ref} = \text{"Depth of Reinf."}$

$\lambda_{s.2.4} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{V.2.4}}{10 \cdot in}}} \right)$ $\quad \lambda_{s.2.4} = 1$ $\quad \lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$\rho_{2.3} = 0.0114$ $\quad \rho_{ref} = \text{"Reinf. Ratio"}$

$Vc_{2.4.a} := 8 \cdot \lambda_{s.2.4} \cdot \lambda \cdot \rho_{2.3}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.a} = 2.158 \cdot kip$

$Vc_{2.4.b} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{strip} \cdot d_{F.2.3} \right)$ $\quad Vc_{2.4.b} = 5.987 \cdot kip$

$\phi_V = 0.75$ $\quad \phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$Vc_{2.4} := \min\left(Vc_{2.4.b}, Vc_{2.4.a} \right)$ $\quad Vc_{2.4} = 2.158 \cdot kip$

$\phi Vc_{2.4} := \phi_V \cdot Vc_{2.4}$ $\quad \phi Vc_{2.4} = 1.618 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Vu_{2.1.mn} = 0 \cdot kip$ $\quad Vu_{ref} = \text{"Ultimate Shear"}$

$chk_{2.6} := \text{if}\left(\phi Vc_{2.4} \geq Vu_{2.1.mn}, \text{"OK"}, \text{"NG"} \right)$ $\quad chk_{2.6} = \text{"OK"}$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref}$ = "Modulus of Rupture, Eqn (19.2.3.1)" $E_c = 2838 \cdot ksi$ $E_{c.ref}$ = "Modulus of Elas. Conc."

$E_{c.spec} = 2991 \cdot ksi$ $E_{c.spec.ref}$ = "Modulus of Elas. Specified Conc."

$Ig_{2.1} := \frac{1}{12} \cdot t_{r.a}^{3} \cdot b_{strip}$ $Ig_{2.1} = 107.172 \cdot in^4$ $I_{g.ref}$ = "Gross Moment of Inertia"

$y_{t.2.1} := \frac{t_{r.a}}{2}$ $y_{t.2.1} = 2.375 \cdot in$ $y_{t.ref}$ = "Dist., Centroid-Tension Face"

$Mcr_{2.1} := f_r \cdot \frac{Ig_{2.1}}{y_{t.2.1}}$ $Mcr_{2.1} = 19.3 \cdot kip \cdot in$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.2.6} := \frac{\left[\left(DL_{r.mx} + w_{llr}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{S.2.6} = 16.101 \cdot kip \cdot in$ $M_{S.ref}$ = "Service Moment"

$chk_{2.7} := if\left(\frac{2}{3} Mcr_{2.1} > M_{S.2.6}, \text{"Cracking not likely"}, \text{"Cracking will occur"}\right)$ $chk_{2.7}$ = "Cracking will occur"

$M_{D.2.6} := \frac{\left[\left(DL_{r.mx}\right) \cdot l_r^{2}\right]}{8} \cdot b_{strip}$ $M_{D.2.6} = 9.748 \cdot kip \cdot in$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{2.1} = 53.586 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1} = -4.899 \cdot in^4$ $0.25 \cdot Ig_{2.1} = 26.793 \cdot in^4$

$I_{c.2.1} := \min\left[0.5 \cdot Ig_{2.1}, \max\left[0.25 \cdot Ig_{2.1}, \left(0.1 + 25 \cdot \rho_{2.1}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{strip}}{d_{F.2.3}}\right) \cdot Ig_{2.1}\right]\right]$ $I_{c.2.1} = 26.79 \cdot in^4$

$I_{cr.slab.ref}$ = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"

$I_{e.D.2.1} := if\left[M_{D.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{D.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.D.2.1} = 107.2 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$I_{e.S.2.1} := if\left[M_{S.2.6} < \frac{2}{3} Mcr_{2.1}, Ig_{2.1}, \frac{I_{c.2.1}}{1 - \left(\frac{\frac{2}{3} Mcr_{2.1}}{M_{S.2.6}}\right)^{2} \cdot \left(1 - \frac{I_{c.2.1}}{Ig_{2.1}}\right)}\right]$ $I_{e.S.2.1} = 51.4 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{D.2.1} := \frac{5 \cdot \left(DL_{r.mx} \cdot b_{strip}\right) \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.D.2.1}}$ $\Delta_{D.2.1} = 0.05519 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.2.1} := \frac{5 \cdot \left[\left(DL_{r.mx} + w_{llr}\right) \cdot b_{strip}\right] \cdot l_r^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.2.1}}$ $\Delta_{S.2.1} = 0.19003 \cdot in$ $\Delta_{Serv.ref}$:= "Serv Load Defl."

$\Delta_{L.2.1} := \Delta_{S.2.1} - \Delta_{D.2.1}$ $\Delta_{L.2.1} = 0.13484 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{S2LL} := \xi_3$ $\xi_{S2LL} = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Live Load

$\xi_{60} = 2$ $\xi_{S2DL} := \xi_{60}$ $\xi_{S2DL} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Dead Load

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$\rho'_{2.1} := 0$ $\quad \rho'_{2.1} = 0$ $\quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\Delta_{LT.2.1} := \Delta_{D.2.1}\cdot\left(1 + \frac{\xi_{S2DL}}{1 + 50\cdot\rho'_{2.1}}\right) + \Delta_{L.2.1}\cdot\left(1 + \frac{\xi_{S2LL}}{1 + 50\cdot\rho'_{2.1}}\right) \qquad \Delta_{LT.2.1} = 0.435\cdot in$$

$\Delta_{LT.ref} = \text{"Time Dep. Defl."}$

$$\Delta_{2.1} := \frac{5\cdot\left[\left(DL_{r.mx} + w_{llr}\right)\cdot l_r^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{2.1}}\cdot b_{strip} \qquad \Delta_{2.1} = 0.091\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.Dam} = 480$ $\quad \eta_{a.RF.Dam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Dam., Tbl (24.2.2)"}$

$\eta_{a.R.Non\_Struct} = 360$ $\quad \eta_{a.R.Non\_Struct.ref} = \text{"Allow Defl Ratio, Flat Roof, Non-Struct, Tbl (24.2.2)"}$

$\eta_{a.RF.NonDam} = 240$ $\quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$

$\eta_{a.2.1} := \eta_{a.RF.NonDam}$ $\quad \eta_{a.2.1} = 240$ $\quad \delta_{2.1} := \frac{l_r}{\eta_{a.2.1}}$ $\quad \delta_{2.1} = 0.55\cdot in$

$\Delta_{2.2} := \Delta_{2.1} + \Delta_{LT.2.1}$ $\quad \Delta_{2.2} = 0.526\cdot in$ $\quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$

$\eta_{2.1} := \frac{l_r}{\Delta_{2.2}}$ $\quad \eta_{2.1} = 250.7$ $\quad \eta_{ref} = \text{"Deflection Ratio"}$

$chk_{2.8} := \text{if}\left(\Delta_{2.2} < \delta_{2.1}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{2.8} = \text{"OK"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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3. WALL DESIGN - ROOF LIVE LOAD OPTION 1 $w_{llr} = 35 \cdot psf$

a. Design Parameters

$t_w = 4 \cdot in$ Min Thickness of Wall Panels

$f'_c = 4500 \cdot psi$

$wgt_{wall.psf} = 39.3 \cdot psf$

$f_y = 60 \cdot ksi$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.667 \end{pmatrix} ft$

$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf$

b. Wiind Pressure Resistance Not adjacent to an Opening $w_{llr} = 35 \cdot psf$

$P_{W.c} = 64.5 \cdot psf$ $GC_{p.cc.max.ref_0}$ = " CC, MAX Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{L.c} = 64.5 \cdot psf$ $GC_{p.cc.max.ref_1}$ = " CC, MIN Ext Pres Coeff, Zone 4, Fig (30.3-1)"

$P_{LC.c} = 79.6 \cdot psf$ $GC_{p.cc.max.ref_2}$ = "CC, MAX Ext Pres Coeff, Zone 5, Fig (30.3-1)"

$W_{3.0} := \begin{pmatrix} \max(|P_{W.c}|, |P_{L.c}|) \\ |P_{LC.c}| \end{pmatrix}$ $W_{3.0} = \begin{pmatrix} 64.45 \\ 79.56 \end{pmatrix} \cdot psf$

$P_{w.ref}$ = "Max Wall Wind Pressure, Main Surface"

$P_{w.corn.ref}$ = "Max Wall Corner Wind Pressure"

$b_{strip} = 12 \cdot in$ $b_{strip.ref}$ = "Strip Width"

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 35 \cdot psf$

$Pu_{3b.h1} = \begin{pmatrix} 1.02 \\ 0.98 \\ 1.21 \\ 1.21 \\ 1.6 \\ 0.98 \\ 1.37 \\ 0.98 \\ 0.85 \\ 0.57 \\ 1.35 \\ 0.63 \\ 0.51 \end{pmatrix} \cdot kip$ $Wu_{3b.h1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 32.2 \\ 32.2 \\ 64.5 \\ 32.2 \\ 10.9 \\ -10.9 \\ 64.5 \\ 64.5 \\ 10.9 \\ -10.9 \end{pmatrix} \cdot plf$ $LRFD = \begin{pmatrix} "1a" & "1.4(D+F)" \\ "2a" & "1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) " \\ "3a" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)" \\ "3b+" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)" \\ "3b-" & "1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)" \\ "4a+" & "1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))" \\ "4a-" & "1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))" \\ "5a+" & "1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)" \\ "5a-" & "1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)" \\ "6a+" & "0.9(D+F)+1.0(W+)+1.6(H)" \\ "6a-" & "0.9(D+F)+1.0(W-)+1.6(H)" \\ "7a+" & "0.9(D+F)+1.0(E+)+1.6(H)" \\ "7a-" & "0.9(D+F)+1.0(E-)+1.6(H)" \end{pmatrix}$

$P_{u.3b.h1} := Pu_{3b.h1}$ $M_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})^2}{8}$ $V_{u.3b.h1} := Wu_{3b.h1} \cdot \frac{(H_{wall_0})}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h1.mx} := \max(P_{u.3b.h1})$ $P_{u.3b.h1.mx} = 1.6 \cdot kip$ $M_{u.3b.h1.mx} := \max(M_{u.3b.h1})$ $M_{u.3b.h1.mx} = 0.65 \cdot kip \cdot ft$

$P_{u.3b.h1.mn} := \min(P_{u.3b.h1})$ $P_{u.3b.h1.mn} = 0.51 \cdot kip$ $M_{u.3b.h1.mn} := \min(M_{u.3b.h1})$ $M_{u.3b.h1.mn} = -0.11 \cdot kip \cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h1.mx} := \max(|V_{u.3b.h1}|)$ $V_{u.3b.h1.mx} = 0.57 \cdot kip$

$P_{u.3b.h1}{}^T = (1.022 \;\; 0.981 \;\; 1.212 \;\; 1.212 \;\; 1.602 \;\; 0.981 \;\; 1.371 \;\; 0.976 \;\; 0.853 \;\; 0.571 \;\; 1.35 \;\; 0.633 \;\; 0.509) \cdot kip$

$M_{u.3b.h1}{}^T = (0 \;\; 0 \;\; 0 \;\; 0.326 \;\; 0.326 \;\; 0.653 \;\; 0.326 \;\; 0.11 \;\; -0.11 \;\; 0.653 \;\; 0.653 \;\; 0.11 \;\; -0.11) \cdot kip \cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF/200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3b.h1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.145 \;\; 0.145 \;\; 0.29 \;\; 0.145 \;\; 0.049 \;\; -0.049 \;\; 0.29 \;\; 0.29 \;\; 0.049 \;\; -0.049)\cdot kip$

Factored Loads for Panel Height $H_{wall_1} = 10ft$ & Roof Live Load $w_{llr} = 35\cdot psf$

$Pu_{3b.h2}^{T} = (1.09 \;\; 1.04 \;\; 1.27 \;\; 1.27 \;\; 1.66 \;\; 1.04 \;\; 1.43 \;\; 1.03 \;\; 0.91 \;\; 0.61 \;\; 1.39 \;\; 0.67 \;\; 0.54)\cdot kip$

$Wu_{3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 32.2 \;\; 32.2 \;\; 64.5 \;\; 32.2 \;\; 10.9 \;\; -10.9 \;\; 64.5 \;\; 64.5 \;\; 10.9 \;\; -10.9)\cdot plf$

$P_{u.3b.h2} := Pu_{3b.h2} \qquad M_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)^2}{8} \qquad V_{u.3b.h2} := Wu_{3b.h2}\cdot\frac{\left(H_{wall_1}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h2.mx} := \max(P_{u.3b.h2}) \quad P_{u.3b.h2.mx} = 1.66\cdot kip \qquad M_{u.3b.h2.mx} := \max(M_{u.3b.h2}) \quad M_{u.3b.h2.mx} = 0.81\cdot kip\cdot ft$

$P_{u.3b.h2.mn} := \min(P_{u.3b.h2}) \quad P_{u.3b.h2.mn} = 0.54\cdot kip \qquad M_{u.3b.h2.mn} := \min(M_{u.3b.h2}) \quad M_{u.3b.h2.mn} = -0.14\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h2.mx} := \max(|V_{u.3b.h2}|) \quad V_{u.3b.h2.mx} = 0.63\cdot kip$

$P_{u.3b.h2}^{T} = (1.086 \;\; 1.036 \;\; 1.267 \;\; 1.267 \;\; 1.656 \;\; 1.036 \;\; 1.425 \;\; 1.031 \;\; 0.907 \;\; 0.606 \;\; 1.386 \;\; 0.668 \;\; 0.544)\cdot kip$

$M_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.403 \;\; 0.403 \;\; 0.806 \;\; 0.403 \;\; 0.136 \;\; -0.136 \;\; 0.806 \;\; 0.806 \;\; 0.136 \;\; -0.136)\cdot kip\cdot ft$

$V_{u.3b.h2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.161 \;\; 0.161 \;\; 0.322 \;\; 0.161 \;\; 0.054 \;\; -0.054 \;\; 0.322 \;\; 0.322 \;\; 0.054 \;\; -0.054)\cdot kip$

Factored Loads for Panel Height $H_{wall_2} = 10.667ft$ & Roof Live Load $w_{llr} = 35\cdot psf$

$Pu_{3b.h3}^{T} = (1.13 \;\; 1.07 \;\; 1.3 \;\; 1.3 \;\; 1.69 \;\; 1.07 \;\; 1.46 \;\; 1.07 \;\; 0.94 \;\; 0.63 \;\; 1.41 \;\; 0.69 \;\; 0.57)\cdot kip$

$Wu_{3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 32.2 \;\; 32.2 \;\; 64.5 \;\; 32.2 \;\; 10.9 \;\; -10.9 \;\; 64.5 \;\; 64.5 \;\; 10.9 \;\; -10.9)\cdot plf$

$P_{u.3b.h3} := Pu_{3b.h3} \qquad M_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)^2}{8} \qquad V_{u.3b.h3} := Wu_{3b.h3}\cdot\frac{\left(H_{wall_2}\right)}{2}$

$P_{u.ref}$ = "Fact. Axial Load" $\qquad M_{u.ref}$ = "Fact. Moment"

$P_{u.3b.h3.mx} := \max(P_{u.3b.h3}) \quad P_{u.3b.h3.mx} = 1.69\cdot kip \qquad M_{u.3b.h3.mx} := \max(M_{u.3b.h3}) \quad M_{u.3b.h3.mx} = 0.92\cdot kip\cdot ft$

$P_{u.3b.h3.mn} := \min(P_{u.3b.h3}) \quad P_{u.3b.h3.mn} = 0.57\cdot kip \qquad M_{u.3b.h3.mn} := \min(M_{u.3b.h3}) \quad M_{u.3b.h3.mn} = -0.15\cdot kip\cdot ft$

$V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3b.h3.mx} := \max(|V_{u.3b.h3}|) \quad V_{u.3b.h3.mx} = 0.68\cdot kip$

$P_{u.3b.h3}^{T} = (1.128 \;\; 1.072 \;\; 1.303 \;\; 1.303 \;\; 1.693 \;\; 1.072 \;\; 1.462 \;\; 1.067 \;\; 0.943 \;\; 0.63 \;\; 1.409 \;\; 0.692 \;\; 0.568)\cdot kip$

$M_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.458 \;\; 0.458 \;\; 0.917 \;\; 0.458 \;\; 0.155 \;\; -0.155 \;\; 0.917 \;\; 0.917 \;\; 0.155 \;\; -0.155)\cdot kip\cdot ft$

$V_{u.3b.h3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.172 \;\; 0.172 \;\; 0.344 \;\; 0.172 \;\; 0.058 \;\; -0.058 \;\; 0.344 \;\; 0.344 \;\; 0.058 \;\; -0.058)\cdot kip$

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

Try $B_{3.0}$ = "WWF.4x4_w4xw4" $\quad S_{3.0}$ = "N/A" $\cdot in \qquad A_{s.3b} = 0.12\cdot\frac{in^2}{ft} \qquad d_{3b} := \frac{t_w}{2} - 0.25in \qquad d_{3b} = 1.75\cdot in$

$\phi_V = 0.75$ $\phi_{V.ref}$ = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ λ_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\phi v_{n.3b} := \phi_V \cdot \lambda \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot \min(d_{3b}, t_w - d_{3b}) \cdot b_{strip}$

$\phi v_{n.3b} = 1.796 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3b} := \max(V_{u.3b.h1.mx}, V_{u.3b.h2.mx}, V_{u.3b.h3.mx})$ $V_{u.3b} = 0.676 \cdot kip$

$chk_{3b.0} := \text{if}(\phi v_{n.3b} > V_{u.3b}, \text{"OK"}, \text{"NG"})$ $chk_{3b.0}$ = "OK"

$I_{3b} := \frac{b_{strip} \cdot t_w^3}{12}$ $I_{3b} = 64 \cdot in^4$ $EI_{3b} := 0.25 \cdot E_c \cdot I_{3b}$ $EI_{3b} = 45400.8 \cdot kip \cdot in^2$ Moment of Inertia/Stiffness of Strip

$k_{wall} := 1.0$ $P_{c.3b} := \frac{\pi^2 \cdot EI_{3b}}{(k_{wall} \cdot H_{wall})^2}$ $P_{c.3b} = \begin{pmatrix} 38.4 \\ 31.1 \\ 27.3 \end{pmatrix} \cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} \cdot ft$

$\delta_{3b.h1} := \frac{1}{1 - \frac{P_{u.3b.h1.mx}}{0.75 \cdot P_{c.3b_0}}}$ $\delta_{3b.h1} = 1.059$ $\Omega_{\delta.3b.h1} := \delta_{3b.h1}^{-1}$ $\Omega_{\delta.3b.h1} = 0.944$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

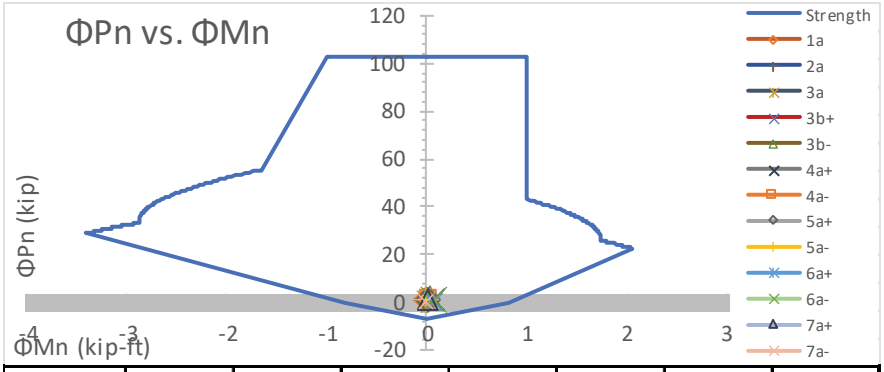
$\delta_{3b.h2} := \frac{1}{1 - \frac{P_{u.3b.h2.mx}}{0.75 \cdot P_{c.3b_1}}}$ $\delta_{3b.h2} = 1.076$ $\Omega_{\delta.3b.h2} := \delta_{3b.h2}^{-1}$ $\Omega_{\delta.3b.h2} = 0.929$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\delta_{3b.h3} := \frac{1}{1 - \frac{P_{u.3b.h3.mx}}{0.75 \cdot P_{c.3b_2}}}$ $\delta_{3b.h3} = 1.09$ $\Omega_{\delta.3b.h3} := \delta_{3b.h3}^{-1}$ $\Omega_{\delta.3b.h3} = 0.917$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$\Omega_{\delta.3b} := \min(\Omega_{\delta.3b.h1}, \Omega_{\delta.3b.h2}, \Omega_{\delta.3b.h3})$ $\Omega_{\delta.3b} = 0.917$



$chk_{3b.1}$ = "OK"

Axial/Flexure Check

LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	1.1%	1.0%	1.3%	3.58%	3.5%	7.3%	3.5%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.2%	1.2%	7.5%	7.1%	1.3%	1.2%	
Check	OK	OK	OK	OK	OK	OK	

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Axial & Flexure @ (Door)

Check walls due to roof load & out-plane wind load

$w_{door} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$ Door Widths

$b_{3c.0} := \frac{w_{door}}{2} + 1\,ft$ $b_{3c.0} = \begin{pmatrix} 4 \\ 3 \\ 2.5 \end{pmatrix} ft$ Tributary Width of Loading

$S_{3c} = \begin{pmatrix} 24 \\ 24 \\ 16 \end{pmatrix} \cdot in$ $S_{3.ref} = "Min.\ Space\ to\ Adjacent\ Opening"$ $b_{3c.1} = \begin{pmatrix} 12 \\ 12 \\ 8 \end{pmatrix} \cdot in$ $b_{3c.1.ref} = "Strip\ Width\ Btwn\ Openings"$

$h_{door} = 7\,ft$ Max Door Opening Height

Wall Panel Reinforcement Pattern(s) and Thickness is Constant For all Wall Panel Heights and Various Roof Live Loads.

$B_{3.1} = "\#6"$ $B_{3.1.ref} = "Wall\ Vert.\ Perim.\ Reinf."$ $B_{3.2} = "\#4"$ $B_{3.2.ref} = "Wall\ Top.\ Perim.\ Reinf."$

$N_{B.3.1} = 1$ $N_{B.3.1.ref} = "\#\ Wall\ Vert.\ Perim.\ Bars"$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = "\#\ Wall\ Top\ Perim.\ Bars"$

$D_{3c.1} = 0.75 \cdot in$ $D_{ref} = "Bar\ Diam."$ $D_{3c.2} = 0.5 \cdot in$ $D_{ref} = "Bar\ Diam."$

$A_{3c.1} = 0.44 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.3} = "\#6"$ $B_{3.3.ref} = "EW\ Bot\ Perim.\ Reinf."$ $B_{3.4} = "\#4"$ $B_{3.4.ref} = "SW\ Bot\ Perim.\ Reinf."$

$N_{B.3.3} = 1$ $N_{B.3.3.ref} = "\#\ EW\ Bot\ Perim.\ Bars"$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = "\#\ SW\ Bot\ Perim.\ Bars"$

$D_{3c.3} = 0.75 \cdot in$ $D_{ref} = "Bar\ Diam."$ $D_{3c.4} = 0.5 \cdot in$ $D_{ref} = "Bar\ Diam."$

$A_{3c.3} = 0.44 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$ $A_{3c.4} = 0.4 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

$B_{3.5} = "\#5"$ $B_{3.5.ref} = "Wall\ Opening\ Reinf."$ $D_{3c.5} = 0.625 \cdot in$ $D_{ref} = "Bar\ Diam."$

$N_{B.3.5} = 2$ $N_{B.3.5.ref} = "\#\ Wall\ Opening\ Bars"$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = "Area\ of\ Reinf."$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ & Roof Live Load $w_{llr} = 35 \cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24 \cdot in$

$W_{u.3c.h1d1}^T = (0 \quad 0 \quad 0 \quad 128.9 \quad 128.9 \quad 257.8 \quad 128.9 \quad 43.5 \quad -43.5 \quad 257.8 \quad 257.8 \quad 43.5 \quad -43.5) \cdot plf$

$P_{u.3c.h1d1}^T = (4.09 \quad 3.93 \quad 4.85 \quad 4.85 \quad 6.41 \quad 3.93 \quad 5.48 \quad 3.91 \quad 3.41 \quad 2.28 \quad 5.4 \quad 2.53 \quad 2.04) \cdot kip$

$M_{u.3c.h1d1}^T = (0 \quad 0 \quad 0 \quad 0.79 \quad 0.79 \quad 1.58 \quad 0.79 \quad 0.27 \quad -0.27 \quad 1.58 \quad 1.58 \quad 0.27 \quad -0.27) \cdot kip \cdot ft$

$V_{u.3c.h1d1}^T = (0 \quad 0 \quad 0 \quad 0.45 \quad 0.45 \quad 0.9 \quad 0.45 \quad 0.15 \quad -0.15 \quad 0.9 \quad 0.9 \quad 0.15 \quad -0.15) \cdot kip$

$P_{u.ref} = "Fact.\ Axial\ Load"$ $M_{u.ref} = "Fact.\ Moment"$ $V_{u.ref} = "Fact.\ Shear\ Load"$

$P_{u.3c.h1d1.mx} = 6.41 \cdot kip$ $M_{u.3c.h1d1.mx} = 1.58 \cdot kip \cdot ft$ $V_{u.3c.h1d1.mx} = 1.77 \cdot kip$

$P_{u.3c.h1d1.mn} = 2.04 \cdot kip$ $M_{u.3c.h1d1.mn} = -0.27 \cdot kip \cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 35 \cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24 \cdot in$

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$W_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 96.7 \;\; 96.7 \;\; 193.4 \;\; 96.7 \;\; 32.6 \;\; -32.6 \;\; 193.4 \;\; 193.4 \;\; 32.6 \;\; -32.6)\cdot plf$

$P_{u.3c.h1d2}^{T} = (3.07 \;\; 2.94 \;\; 3.64 \;\; 3.64 \;\; 4.81 \;\; 2.94 \;\; 4.11 \;\; 2.93 \;\; 2.56 \;\; 1.71 \;\; 4.05 \;\; 1.9 \;\; 1.53)\cdot kip$

$M_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.59 \;\; 0.59 \;\; 1.18 \;\; 0.59 \;\; 0.2 \;\; -0.2 \;\; 1.18 \;\; 1.18 \;\; 0.2 \;\; -0.2)\cdot kip\cdot ft$

$V_{u.3c.h1d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.34 \;\; 0.34 \;\; 0.68 \;\; 0.34 \;\; 0.11 \;\; -0.11 \;\; 0.68 \;\; 0.68 \;\; 0.11 \;\; -0.11)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d2.mx} = 4.81\cdot kip$ $M_{u.3c.h1d2.mx} = 1.18\cdot kip\cdot ft$ $V_{u.3c.h1d2.mx} = 1.33\cdot kip$

$P_{u.3c.h1d2.mn} = 1.53\cdot kip$ $M_{u.3c.h1d2.mn} = -0.2\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_0} = 9\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 80.6 \;\; 80.6 \;\; 161.1 \;\; 80.6 \;\; 27.2 \;\; -27.2 \;\; 161.1 \;\; 161.1 \;\; 27.2 \;\; -27.2)\cdot plf$

$P_{u.3c.h1d3}^{T} = (2.56 \;\; 2.45 \;\; 3.03 \;\; 3.03 \;\; 4.01 \;\; 2.45 \;\; 3.43 \;\; 2.44 \;\; 2.13 \;\; 1.43 \;\; 3.38 \;\; 1.58 \;\; 1.27)\cdot kip$

$M_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.99 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.99 \;\; 0.99 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h1d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h1d3.mx} = 4.01\cdot kip$ $M_{u.3c.h1d3.mx} = 0.99\cdot kip\cdot ft$ $V_{u.3c.h1d3.mx} = 1.11\cdot kip$

$P_{u.3c.h1d3.mn} = 1.27\cdot kip$ $M_{u.3c.h1d3.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 128.9 \;\; 128.9 \;\; 257.8 \;\; 128.9 \;\; 43.5 \;\; -43.5 \;\; 257.8 \;\; 257.8 \;\; 43.5 \;\; -43.5)\cdot plf$

$P_{u.3c.h2d1}^{T} = (4.34 \;\; 4.14 \;\; 5.07 \;\; 5.07 \;\; 6.63 \;\; 4.14 \;\; 5.7 \;\; 4.12 \;\; 3.63 \;\; 2.42 \;\; 5.54 \;\; 2.67 \;\; 2.18)\cdot kip$

$M_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.79 \;\; 0.79 \;\; 1.58 \;\; 0.79 \;\; 0.27 \;\; -0.27 \;\; 1.58 \;\; 1.58 \;\; 0.27 \;\; -0.27)\cdot kip\cdot ft$

$V_{u.3c.h2d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.45 \;\; 0.45 \;\; 0.9 \;\; 0.45 \;\; 0.15 \;\; -0.15 \;\; 0.9 \;\; 0.9 \;\; 0.15 \;\; -0.15)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d1.mx} = 6.63\cdot kip$ $M_{u.3c.h2d1.mx} = 1.58\cdot kip\cdot ft$ $V_{u.3c.h2d1.mx} = 1.77\cdot kip$

$P_{u.3c.h2d1.mn} = 2.18\cdot kip$ $M_{u.3c.h2d1.mn} = -0.27\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h21d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 96.7 \;\; 96.7 \;\; 193.4 \;\; 96.7 \;\; 32.6 \;\; -32.6 \;\; 193.4 \;\; 193.4 \;\; 32.6 \;\; -32.6)\cdot plf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$P_{u.3c.h2d2}^{T} = (3.26 \;\; 3.11 \;\; 3.8 \;\; 3.8 \;\; 4.97 \;\; 3.11 \;\; 4.28 \;\; 3.09 \;\; 2.72 \;\; 1.82 \;\; 4.16 \;\; 2 \;\; 1.63)\cdot kip$

$M_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.59 \;\; 0.59 \;\; 1.18 \;\; 0.59 \;\; 0.2 \;\; -0.2 \;\; 1.18 \;\; 1.18 \;\; 0.2 \;\; -0.2)\cdot kip\cdot ft$

$V_{u.3c.h2d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.34 \;\; 0.34 \;\; 0.68 \;\; 0.34 \;\; 0.11 \;\; -0.11 \;\; 0.68 \;\; 0.68 \;\; 0.11 \;\; -0.11)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d2.mx} = 4.97\cdot kip$ $M_{u.3c.h2d2.mx} = 1.18\cdot kip\cdot ft$ $V_{u.3c.h2d2.mx} = 1.33\cdot kip$

$P_{u.3c.h2d2.mn} = 1.63\cdot kip$ $M_{u.3.1.h2r2d2.mn} = -0.2\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_1} = 10\,ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 80.6 \;\; 80.6 \;\; 161.1 \;\; 80.6 \;\; 27.2 \;\; -27.2 \;\; 161.1 \;\; 161.1 \;\; 27.2 \;\; -27.2)\cdot plf$

$P_{u.3.1.h2d3}^{T} = (2.71 \;\; 2.59 \;\; 3.17 \;\; 3.17 \;\; 4.14 \;\; 2.59 \;\; 3.56 \;\; 2.58 \;\; 2.27 \;\; 1.52 \;\; 3.46 \;\; 1.67 \;\; 1.36)\cdot kip$

$M_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.99 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.99 \;\; 0.99 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h2d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h2d3.mx} = 4.14\cdot kip$ $M_{u.3c.h2d3.mx} = 0.99\cdot kip\cdot ft$ $V_{u.3c.h2d3.mx} = 1.11\cdot kip$

$P_{u.3c.h2d3.mn} = 1.36\cdot kip$ $M_{u.3.1.h2d3.mn} = -0.17\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_0} = 6\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_0} = 24\cdot in$

$W_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 128.9 \;\; 128.9 \;\; 257.8 \;\; 128.9 \;\; 43.5 \;\; -43.5 \;\; 257.8 \;\; 257.8 \;\; 43.5 \;\; -43.5)\cdot plf$

$P_{u.3c.h3d1}^{T} = (4.51 \;\; 4.29 \;\; 5.21 \;\; 5.21 \;\; 6.77 \;\; 4.29 \;\; 5.85 \;\; 4.27 \;\; 3.77 \;\; 2.52 \;\; 5.64 \;\; 2.77 \;\; 2.27)\cdot kip$

$M_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.79 \;\; 0.79 \;\; 1.58 \;\; 0.79 \;\; 0.27 \;\; -0.27 \;\; 1.58 \;\; 1.58 \;\; 0.27 \;\; -0.27)\cdot kip\cdot ft$

$V_{u.3c.h3d1}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.45 \;\; 0.45 \;\; 0.9 \;\; 0.45 \;\; 0.15 \;\; -0.15 \;\; 0.9 \;\; 0.9 \;\; 0.15 \;\; -0.15)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d1.mx} = 6.77\cdot kip$ $M_{u.3c.h3d1.mx} = 1.58\cdot kip\cdot ft$ $V_{u.3c.h3d1.mx} = 1.77\cdot kip$

$P_{u.3c.h3d1.mn} = 2.27\cdot kip$ $M_{u.3c.h3d1.mn} = -0.27\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_1} = 4\,ft$ & Minimum Space between Door and Next Opening is >= $S_{3c_1} = 24\cdot in$

$W_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 96.7 \;\; 96.7 \;\; 193.4 \;\; 96.7 \;\; 32.6 \;\; -32.6 \;\; 193.4 \;\; 193.4 \;\; 32.6 \;\; -32.6)\cdot plf$

$P_{u.3c.h3d2}^{T} = (3.38 \;\; 3.22 \;\; 3.91 \;\; 3.91 \;\; 5.08 \;\; 3.22 \;\; 4.39 \;\; 3.2 \;\; 2.83 \;\; 1.89 \;\; 4.23 \;\; 2.08 \;\; 1.7)\cdot kip$

$M_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.59 \;\; 0.59 \;\; 1.18 \;\; 0.59 \;\; 0.2 \;\; -0.2 \;\; 1.18 \;\; 1.18 \;\; 0.2 \;\; -0.2)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$V_{u.3c.h3d2}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.34 \;\; 0.34 \;\; 0.68 \;\; 0.34 \;\; 0.11 \;\; -0.11 \;\; 0.68 \;\; 0.68 \;\; 0.11 \;\; -0.11)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d2.mx} = 5.08\cdot kip$ $M_{u.3c.h3d2.mx} = 1.18\cdot kip\cdot ft$ $V_{u.3c.h3d2.mx} = 1.33\cdot kip$

$P_{u.3c.h3d2.mn} = 1.7\cdot kip$ $M_{u.3c.h3d2.mn} = -0.2\cdot kip\cdot ft$

Factored Loads for Panel Height $H_{wall_2} = 10.67ft$ Roof Live Load $w_{llr} = 35\cdot psf$ & Door Width Less <= $w_{door_2} = 3\,ft$

& Minimum Space between Door and Next Opening is >= $S_{3c_2} = 16\cdot in$

$W_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 80.6 \;\; 80.6 \;\; 161.1 \;\; 80.6 \;\; 27.2 \;\; -27.2 \;\; 161.1 \;\; 161.1 \;\; 27.2 \;\; -27.2)\cdot plf$

$P_{u.3.1.h3r2d3}^{T} = \blacksquare\cdot kip$

$M_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.49 \;\; 0.49 \;\; 0.99 \;\; 0.49 \;\; 0.17 \;\; -0.17 \;\; 0.99 \;\; 0.99 \;\; 0.17 \;\; -0.17)\cdot kip\cdot ft$

$V_{u.3c.h3d3}^{T} = (0 \;\; 0 \;\; 0 \;\; 0.28 \;\; 0.28 \;\; 0.56 \;\; 0.28 \;\; 0.1 \;\; -0.1 \;\; 0.56 \;\; 0.56 \;\; 0.1 \;\; -0.1)\cdot kip$

$P_{u.ref}$ = "Fact. Axial Load" $M_{u.ref}$ = "Fact. Moment" $V_{u.ref}$ = "Fact. Shear Load"

$P_{u.3c.h3d3.mx} = 4.23\cdot kip$ $M_{u.3c.h3d3.mx} = 0.99\cdot kip\cdot ft$ $V_{u.3c.h3d3.mx} = 1.11\cdot kip$

$P_{u.3c.h3d3.mn} = 1.42\cdot kip$ $M_{u.3c.h3d3.mn} = -0.17\cdot kip\cdot ft$

Try $A_{s.3b}\cdot b_{3c.1} = \begin{pmatrix} 0.12 \\ 0.12 \\ 0.08 \end{pmatrix}\cdot in^2$ Wall Vert. Area Reinf. $A_{3c.5} = 0.62\cdot in^2$ Min Vertical Opening/Perim. Reinf.

$A_{S.3c} := A_{s.3b}\cdot b_{3c.1} + A_{3c.5}$ $A_{S.3c} = \begin{pmatrix} 0.74 \\ 0.74 \\ 0.7 \end{pmatrix}\cdot in^2$ $d_{3c} := \min\left[d_{3b}, t_w - \left(0.75in + 0.5D_{3c.5}\right)\right]$ $d_{3c} = 1.75\cdot in$ Depth/Area of Opening Rebar

$\phi v_{n.3c} := \phi_V\cdot\lambda\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot d_{3c}\cdot b_{3c.1}$ $\phi v_{n.3c} = \begin{pmatrix} 1.796 \\ 1.796 \\ 1.197 \end{pmatrix}\cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$V_{u.3c.d1} = 1.774\cdot kip$ Max for Door 1 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d2} = 1.33\cdot kip$ Max for Door 2 $V_{u.ref}$ = "Fact. Shear Load"

$V_{u.3c.d3} = 1.109\cdot kip$ Max for Door 3 $V_{u.ref}$ = "Fact. Shear Load"

$chk_{3c.0} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.0}$ = "OK"

$chk_{3c.1} := \text{if}\left(\phi v_{n.3c_1} > V_{u.3c.d2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.1}$ = "OK"

$chk_{3c.2} := \text{if}\left(\phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3c.2}$ = "OK"

$chk_{3c} := \text{if}\left(\phi v_{n.3c_0} > V_{u.3c.d1} \wedge \phi v_{n.3c_1} > V_{u.3c.d2} \wedge \phi v_{n.3c_2} > V_{u.3c.d3}, \text{"OK"}, \text{"NG"}\right)$ chk_{3c} = "OK"

$I_{3c} := \dfrac{b_{3c.1}\cdot t_w^{3}}{12}$ $I_{3c} = \begin{pmatrix} 64 \\ 64 \\ 42.7 \end{pmatrix}\cdot in^4$ $EI_{3c} := 0.25\cdot E_c\cdot I_{3c}$ $EI_{3c} = \begin{pmatrix} 45400.8 \\ 45400.8 \\ 30267.2 \end{pmatrix}\cdot kip\cdot in^2$ Moment of Inertia/Stiffness of Strip Width

$k_{wall} = 1$ $P_{c.3c} := \dfrac{\pi^2\cdot EI_{3c}}{\left(k_{wall}\cdot H_{wall}\right)^2}$ $P_{c.3c} = \begin{pmatrix} 38.4 \\ 31.1 \\ 18.2 \end{pmatrix}\cdot kip$ Eulier Buckling Load $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}\cdot ft$

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$$\delta_{3c.h1d1} := \frac{1}{1 - \frac{P_{u.3c.h1d1.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.286 \qquad \Omega_{\delta.3c.h1d1} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d1} = 0.778$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

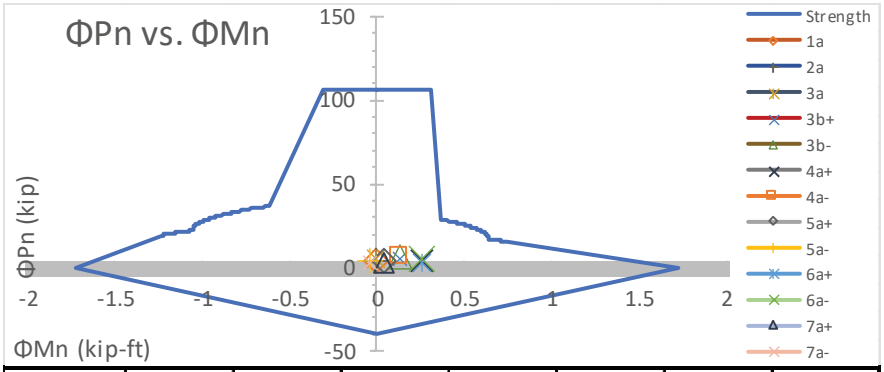
$$\delta_{3c.h2d1} := \frac{1}{1 - \frac{P_{u.3c.h2d1.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d1} = 1.396 \qquad \Omega_{\delta.3c.h2d1} := \delta_{3c.h2d1}^{-1} \qquad \Omega_{\delta.3c.h2d1} = 0.716$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h3d1} := \frac{1}{1 - \frac{P_{u.3c.h3d1.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d1} = 1.981 \qquad \Omega_{\delta.3c.h3d1} := \delta_{3c.h3d1}^{-1} \qquad \Omega_{\delta.3c.h3d1} = 0.505$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67ft$ & Door 1 $w_{door_0} = 6 \cdot ft$

$$\Omega_{\delta.3c.d1} := \min\left(\Omega_{\delta.3c.h1d1}, \Omega_{\delta.3c.h2d1}, \Omega_{\delta.3c.h3d1}\right) \qquad \Omega_{\delta.3c.d1} = 0.505$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	4.2%	4.0%	4.9%	9.34%	10.1%	17.9%	9.6%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	4.0%	3.5%	16.6%	19.1%	2.8%	2.6%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.3}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$

Door width <= $w_{door_0} = 6 \cdot ft$

$$\delta_{3c.h1d2} := \frac{1}{1 - \frac{P_{u.3c.h1d2.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d1} = 1.286 \qquad \Omega_{\delta.3c.h1d2} := \delta_{3c.h1d1}^{-1} \qquad \Omega_{\delta.3c.h1d2} = 0.778$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h2d2} := \frac{1}{1 - \frac{P_{u.3c.h2d2.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d2} = 1.271 \qquad \Omega_{\delta.3c.h2d2} := \delta_{3c.h2d2}^{-1} \qquad \Omega_{\delta.3c.h2d2} = 0.787$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h3d2} := \frac{1}{1 - \frac{P_{u.3c.h3d2.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d2} = 1.591 \qquad \Omega_{\delta.3c.h3d2} := \delta_{3c.h3d2}^{-1} \qquad \Omega_{\delta.3c.h3d2} = 0.629$$

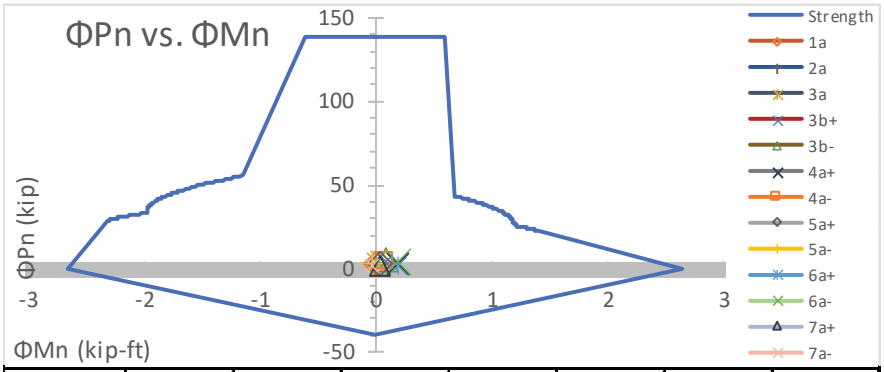
Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_1} = 4 \cdot ft$

$$\Omega_{\delta.3c.d2} := \min\left(\Omega_{\delta.3c.h1d2}, \Omega_{\delta.3c.h2d2}, \Omega_{\delta.3c.h3d2}\right) \qquad \Omega_{\delta.3c.d2} = 0.629$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.4%	2.3%	2.8%	3.98%	4.1%	7.8%	4.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	2.3%	2.0%	7.6%	8.0%	1.5%	1.2%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.4}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_1} = 4 \cdot ft$

$$\delta_{3c.h1d3} := \frac{1}{1 - \frac{P_{u.3c.h1d3.mx}}{0.75 \cdot P_{c.3c_0}}} \qquad \delta_{3c.h1d3} = 1.161 \qquad \Omega_{\delta.3c.h1d3} := \delta_{3c.h1d3}^{-1} \qquad \Omega_{\delta.3c.h1d3} = 0.861$$

Amp Factor / Red. Srength Panel Height $H_{wall_0} = 9\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

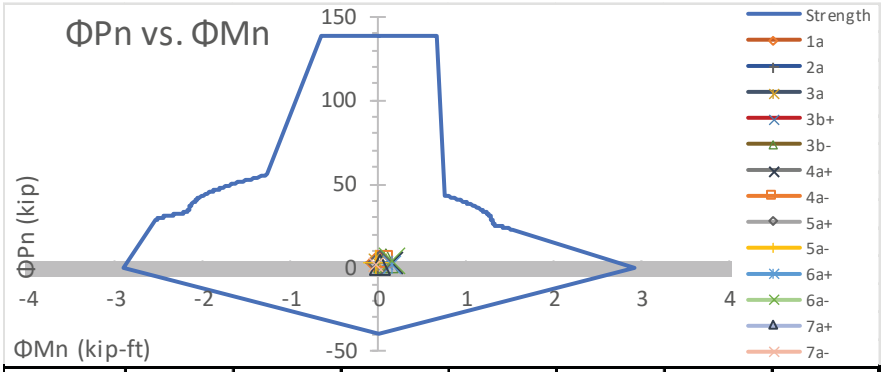
$$\delta_{3c.h2d3} := \frac{1}{1 - \frac{P_{u.3c.h2d3.mx}}{0.75 \cdot P_{c.3c_1}}} \qquad \delta_{3c.h2d3} = 1.216 \qquad \Omega_{\delta.3c.h2d3} := \delta_{3c.h2d3}^{-1} \qquad \Omega_{\delta.3c.h2d3} = 0.823$$

Amp Factor / Red. Srength Panel Height $H_{wall_1} = 10\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\delta_{3c.h3d3} := \frac{1}{1 - \frac{P_{u.3c.h3d3.mx}}{0.75 \cdot P_{c.3c_2}}} \qquad \delta_{3c.h3d3} = 1.448 \qquad \Omega_{\delta.3c.h3d3} := \delta_{3c.h3d3}^{-1} \qquad \Omega_{\delta.3c.h3d3} = 0.691$$

Amp Factor / Red. Srength Panel Height $H_{wall_2} = 10.67\,ft$ & Door 1 $w_{door_2} = 3 \cdot ft$

$$\Omega_{\delta.3c.d3} := \min\left(\Omega_{\delta.3c.h1d3}, \Omega_{\delta.3c.h2d3}, \Omega_{\delta.3c.h3d3}\right) \qquad \Omega_{\delta.3c.d3} = 0.691$$



LC	1a	2a	3a	3b+	3b-	4a+	4a-
%Unity	2.0%	1.9%	2.4%	2.97%	3.1%	5.9%	3.0%
Check	OK	OK	OK	OK	OK	OK	OK
LC	5a+	5a-	6a+	6a-	7a+	7a-	
%Unity	1.9%	1.7%	5.7%	6.0%	1.2%	1.0%	
Check	OK	OK	OK	OK	OK	OK	

$chk_{3c.5}$ = "OK"

Axial/Flexure Check

Panel Heights $H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix}$ ft

Door width <= $w_{door_2} = 3 \cdot ft$

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d. Wall - Header Check (Above Opening) $w_{llr} = 35 \cdot psf$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3c.2} = 0.5 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.2} = 0.4 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3c.5} = 0.625 \cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3c.5} = 0.62 \cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$$h_{3d} = \begin{pmatrix} 40 \\ 21 \\ 17 \end{pmatrix} \cdot in \quad \text{Min Header Height for Opening Widths of} \quad w_{open} = \begin{pmatrix} 6 \\ 4 \\ 3 \end{pmatrix} ft$$

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_0} = 6\,ft$

$$Wu_{3d.0}^{T} = (662.7 \quad 673 \quad 904 \quad 904 \quad 1293.8 \quad 673 \quad 1062.8 \quad 668 \quad 544.3 \quad 370.2 \quad 1149.7 \quad 432.1 \quad 308.4) \cdot plf$$

$wu_{3d.0mx} := \max(Wu_{3d.0})$ $wu_{3d.0mx} = 1293.8 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

$wu_{3d.0mn} := \min(Wu_{3d.0}, 0plf)$ $wu_{3d.0mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

Maximum Flexure

$$d_{3d.0} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ 0.5 \cdot h_{3d_0} \\ h_{3d_0} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_0} - 2in - 0.5D_{3c.5} \end{pmatrix} \qquad A_{3d.0} := \begin{pmatrix} \frac{A_{3c.2}}{N_{B.3.2}} \\ \frac{A_{3c.2}}{N_{B.3.2}} \\ A_{s.3b} \cdot 1ft \\ \frac{A_{3c.5}}{N_{B.3.5}} \\ \frac{A_{3c.5}}{N_{B.3.5}} \end{pmatrix}$$

$$d_{3d.0} = \begin{pmatrix} 2.75 \\ 3.75 \\ 20 \\ 35.06 \\ 37.69 \end{pmatrix} \cdot in \qquad A_{3d.0} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for for:
$w_{llr} = 35 \cdot psf$ & $w_{open_0} = 6\,ft$

$P := 0kip$ Applied Axial Load (Reduce Cracking Effect)

given $a_{3d.0.g} := 3.32348\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.0}(a_{3d.0.g}) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0.g} + P \ldots$$
$$+ A_{3d.0_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_3}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.0_4} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.0.g} \cdot \beta_1} \cdot d_{3d.0_4}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.0} := \text{root}(f_{3d.0}(a_{3d.0.g}), a_{3d.0.g})$

$a_{3d.0} = 3.32348 \cdot in$

Depth of Compression Block

$c_{3d.0} := \frac{a_{3d.0}}{\beta_1}$ $c_{3d.0} = 4.028 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.0} + P = -50.8 \cdot kip$

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$F1_{3d.0} := A_{3d.0_0}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{3d.0} = 0.1\cdot kip$$

$$F2_{3d.0} := A_{3d.0_1}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_1}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{3d.0} = 6.4\cdot kip$$

$$F3_{3d.0} := A_{3d.0_2}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_2}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{3d.0} = 7.2\cdot kip$$

$$F4_{3d.0} := A_{3d.0_3}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_3}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F4_{3d.0} = 18.6\cdot kip$$

$$F5_{3d.0} := A_{3d.0_4}\cdot\max\left[\min\left[\left(-0.003+\frac{0.003}{a_{3d.0}\cdot\beta_1}\cdot d_{3d.0_4}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F5_{3d.0} = 18.6\cdot kip$$

$$-0.85\cdot f'_c\cdot t_w\cdot a_{3d.0} + F1_{3d.0} + F2_{3d.0} + F3_{3d.0} + F4_{3d.0} + F5_{3d.0} = 0\cdot kip$$

$$Mn_{3d.0} := -0.85\cdot f'_c\cdot t_w\cdot a_{3d.0}\cdot\frac{a_{3d.0}}{2} + F1_{3d.0}\cdot d_{3d.0_0} + F2_{3d.0}\cdot d_{3d.0_1} \ldots + F3_{3d.0}\cdot d_{3d.0_2} + F4_{3d.0}\cdot d_{3d.0_3} + F5_{3d.0}\cdot d_{3d.0_4} \qquad Mn_{3d.0} = 119.7\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{3d.0} := -0.003+\frac{0.003}{c_{3d.0}}\cdot d_{3d.0_3} \qquad \varepsilon_{3d.0} = 0.0231 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{3d.0} := \min\left[\max\left[0.65, 0.65+\left(\varepsilon_{3d.0}-0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{3d.0} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{3d.0} := \phi_{3d.0}\cdot Mn_{3d.0} \qquad \phi Mn_{3d.0} = 107.76\cdot k\cdot ft \qquad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$$

$$\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3} = 0.43\cdot in^2$$

$$\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right) = 27.531\cdot in$$

$$\rho_{3d.0} := \frac{\text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} > a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right) + A_{3d.0_3}}{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0}, \text{mean}\left(d_{3d.0_2}, d_{3d.0_3}\right), d_{3d.0_3}\right)\right)\cdot t_w} \qquad \rho_{3d.0} = 0.0039$$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{3d.0} := \sum\begin{pmatrix}\text{if}\left(d_{3d.0_0} = 0in, 0in^2, \text{if}\left(d_{3d.0_0} < a_{3d.0}, A_{3d.0_0}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_1} = 0in, 0in^2, \text{if}\left(d_{3d.0_1} < a_{3d.0}, A_{3d.0_1}, 0in^2\right)\right)\\ \text{if}\left(d_{3d.0_2} = 0in, 0in^2, \text{if}\left(d_{3d.0_2} < a_{3d.0}, A_{3d.0_2}, 0in^2\right)\right)\end{pmatrix}\cdot\left[\frac{1}{\left(t_w\cdot d_{3d.0_0}\right)}\right] \qquad \rho'_{3d.0} = 0.0182$$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\lambda_{s.3d.0} := \min\left(1, \sqrt{\frac{2}{1+\frac{\text{if}\left(d_{3d.0_2} = 0in, d_{3d.0_3}, \text{if}\left(d_{3d.0_2} > a_{3d.0} \wedge a_{3d.0} < d_{3d.0_3}, d_{3d.0_2}, d_{3d.0_3}\right)\right)}{10\cdot in}}}\right) \qquad \lambda_{s.3d.0} = 0.816$$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$$Vc_{3d.0.a} := 8\cdot\lambda_{s.3d.0}\cdot\lambda\cdot\rho_{3d.0}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3} \qquad Vc_{3d.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{3d.0_3}$$

$Vc_{3d.0.a} = 8.23\cdot kip$

$Vc_{3d.0.b} = 39.99\cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi Vc_{3d.0} := \phi_V \cdot \min\left(Vc_{3d.0.a}, Vc_{3d.0.b}\right)$ $\phi Vc_{3d.0} = 6.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$Mu_{3d.0} := \dfrac{wu_{3d.0mx} \cdot \left(w_{open_0}\right)^2}{8}$ $Mu_{3d.0} = 5.82 \cdot kip \cdot ft$

$Vu_{3d.0} := \dfrac{\max\left(\left|wu_{3d.0mx}\right|, \left|wu_{3d.0mn}\right|\right) \cdot w_{open_0}}{2}$ $Vu_{3d.0} = 3.88 \cdot kip$

$chk_{3d.1} := \text{if}\left(\phi Mn_{3d.0} \geq Mu_{3d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{3d.1}$ = "OK"

$chk_{3d.2} :=$ "OK" if $\phi Vc_{3d.0} \cdot 0.5 \geq Vu_{3d.0}$; "Min. Stirrups Req'd" if $\phi Vc_{3d.0} \cdot 0.5 < Vu_{3d.0} \wedge \phi Vc_{3d.0} \geq Vu_{3d.0}$; "Stirrups Req'd" otherwise

$chk_{3d.2}$ = "Min. Stirrups Req'd"

$\phi Vs_{3d.0} := \phi_V \cdot A_{s.3b} \cdot \dfrac{4in}{12in} \cdot \left[f_y \cdot \left(0.5 \cdot h_{3d_0}\right)\right]$ $\phi Vs_{3d.0} = 3 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.0} := \phi Vc_{3d.0} + \phi Vs_{3d.0}$ $\phi Vn_{3d.0} = 9.169 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.3} := \text{if}\left(chk_{3d.2} = \text{"OK"}, \text{"OK"}, \text{if}\left(chk_{3d.2} = \text{"Stirrups Req'd"}, \text{if}\left(\phi Vn_{3d.0} < 2 \cdot Vu_{3d.0}, \text{"NG"}, \text{"OK"}\right), \text{"OK"}\right)\right)$

$chk_{3d.3}$ = "OK"

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_1} = 4\,ft$

$Wu_{3d.1}^T = (562.2 \quad 586.9 \quad 817.9 \quad 817.9 \quad 1207.7 \quad 586.9 \quad 976.7 \quad 581.9 \quad 458.1 \quad 314.2 \quad 1093.7 \quad 376 \quad 252.3) \cdot plf$

$wu_{3d.1mx} := \max\left(Wu_{3d.1}\right)$ $wu_{3d.1mx} = 1207.7 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

$wu_{3d.1mn} := \min\left(Wu_{3d.1}, 0plf\right)$ $wu_{3d.1mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

$$d_{3d.1} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_1} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_1} - 2in - 0.5D_{3c.5} \end{pmatrix}$$

$A_{3d.1} := A_{3d.0}$ $d_{3d.1} = \begin{pmatrix} 2.75 \\ 3.75 \\ 16.06 \\ 18.69 \end{pmatrix} \cdot in$ $A_{3d.1} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Area & Depth of Reinf. for: $w_{llr} = 35 \cdot psf$ & $w_{open_1} = 4\,ft$

given $a_{3d.1.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.1}\left(a_{3d.1.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1.g} + P \ldots + A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots + A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1.g} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.1} := \text{root}\left(f_{3d.1}\left(a_{3d.1.g}\right), a_{3d.1.g}\right)$

$a_{3d.1} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.1} := \dfrac{a_{3d.1}}{\beta_1}$ $c_{3d.1} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.1} := A_{3d.1_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_0}\right) \cdot E_s, f_y\right], -f_y\right]$$

$F1_{3d.1} = 4 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$F2_{3d.1} := A_{3d.1_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_1}\right) \cdot E_s, f_y\right], -f_y\right]$ $F2_{3d.1} = 11.7 \cdot kip$

$F3_{3d.1} := A_{3d.1_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_2}\right) \cdot E_s, f_y\right], -f_y\right]$ $F3_{3d.1} = 7.2 \cdot kip$

$F4_{3d.1} := A_{3d.1_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.1} \cdot \beta_1} \cdot d_{3d.1_3}\right) \cdot E_s, f_y\right], -f_y\right]$ $F4_{3d.1} = 18.6 \cdot kip$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} + F1_{3d.1} + F1_{3d.1} + F3_{3d.1} + F4_{3d.1} = -7.772 \cdot kip$

$Mn_{3d.1} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.1} \cdot \frac{a_{3d.1}}{2} + F1_{3d.1} \cdot d_{3d.1_0} + F2_{3d.1} \cdot d_{3d.1_1} \ldots + F3_{3d.1} \cdot d_{3d.1_2} + F4_{3d.1} \cdot d_{3d.1_3}$ $Mn_{3d.1} = 38.5 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.1} := -0.003 + \frac{0.003}{c_{3d.1}} \cdot d_{3d.1_3}$ $\varepsilon_{3d.1} = 0.014$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.1} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.1} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{3d.1} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.1} := \phi_{3d.1} \cdot Mn_{3d.1}$ $\phi Mn_{3d.1} = 34.64 \cdot k \cdot ft$ $Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) = 17.375 \cdot in$

$\rho_{3d.1} := \frac{\text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} > a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) + A_{3d.1_3}}{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1}, \text{mean}\left(d_{3d.1_2}, d_{3d.1_3}\right), d_{3d.1_3}\right)\right) \cdot t_w}$ $\rho_{3d.1} = 0.0062$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.1} := \sum \begin{pmatrix} \text{if}\left(d_{3d.1_0} = 0in, 0in^2, \text{if}\left(d_{3d.1_0} < a_{3d.1}, A_{3d.1_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_1} = 0in, 0in^2, \text{if}\left(d_{3d.1_1} < a_{3d.1}, A_{3d.1_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.1_2} = 0in, 0in^2, \text{if}\left(d_{3d.1_2} < a_{3d.1}, A_{3d.1_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.1_0}\right)}\right]$ $\rho'_{3d.1} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.1} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.1_2} = 0in, d_{3d.1_3}, \text{if}\left(d_{3d.1_2} > a_{3d.1} \wedge a_{3d.1} < d_{3d.1_3}, d_{3d.1_2}, d_{3d.1_3}\right)\right)}{10 \cdot in}}}\right)$ $\lambda_{s.3d.1} = 0.876$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.1.a} := 8 \cdot \lambda_{s.3d.1} \cdot \lambda \cdot \rho_{3d.1}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.1_3}$ $Vc_{3d.1.a} = 5.48 \cdot kip$

$Vc_{3d.1.b} = 21.31 \cdot kip$

$\phi Vc_{3d.1} := \phi_V \cdot \min\left(Vc_{3d.1.a}, Vc_{3d.1.b}\right)$ $\phi Vc_{3d.1} = 4.1 \cdot kip$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.1} := \frac{wu_{3d.1mx} \cdot \left(w_{open_1}\right)^2}{8}$ $Mu_{3d.1} = 2.42 \cdot kip \cdot ft$ $Vu_{3d.1} := \frac{\max\left(\left|wu_{3d.1mx}\right|, \left|wu_{3d.1mn}\right|\right) \cdot w_{open_1}}{2}$ $Vu_{3d.1} = 2.42 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$chk_{3d.5} := if\left(\phi Mn_{3d.1} \geq Mu_{3d.1}, "OK", "NG"\right)$ $chk_{3d.5} = "OK"$

$chk_{3d.6} := \begin{cases} "OK" & \text{if } \phi Vc_{3d.1} \cdot 0.5 \geq Vu_{3d.1} \\ "Min.\ Stirrups\ Req'd" & \text{if } \phi Vc_{3d.1} \cdot 0.5 < Vu_{3d.1} \wedge \phi Vc_{3d.1} \geq Vu_{3d.1} \\ "Stirrups\ Req'd" & \text{otherwise} \end{cases}$ $chk_{3d.6} = "Min.\ Stirrups\ Req'd"$

$\phi Vs_{3d.1} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left[f_y \cdot \left(0.5\, h_{3d_1}\right)\right]$ $\phi Vs_{3d.1} = 1.57 \cdot kip$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.1} := \phi Vc_{3d.1} + \phi Vs_{3d.1}$ $\phi Vn_{3d.1} = 5.688 \cdot kip$ $\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$chk_{3d.7} := if\left(chk_{3d.6} = "OK", "OK", if\left(chk_{3d.6} = "Stirrups\ Req'd", if\left(\phi Vn_{3d.1} < 2 \cdot Vu_{3d.1}, "NG", "OK"\right), "OK"\right)\right)$ $chk_{3d.7} = "OK"$

Roof Live Load $w_{llr} = 35 \cdot psf$ & Opening Width Less <= $w_{open_2} = 3\,ft$

$Wu_{3d.2}^{T} = (541.1 \quad 568.8 \quad 799.8 \quad 799.8 \quad 1189.5 \quad 568.8 \quad 958.5 \quad 563.7 \quad 440 \quad 302.4 \quad 1081.9 \quad 364.2 \quad 240.5) \cdot plf$

$wu_{3d.2mx} := \max\left(Wu_{3d.2}\right)$ $wu_{3d.2mx} = 1189.5 \cdot plf$ Max. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

$wu_{3d.2mn} := \min\left(Wu_{3d.2}, 0plf\right)$ $wu_{3d.2mn} = 0 \cdot plf$ Min. Load on Header for $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

$$d_{3d.2} := \begin{pmatrix} 2.5in + 0.5D_{3c.2} \\ 3.5in + 0.5D_{3c.2} \\ h_{3d_2} - 2in - D_{3c.5} - 2in - 0.5D_{3c.5} \\ h_{3d_2} - 2in - 0.5D_{3c.5} \end{pmatrix} \quad A_{3d.2} := A_{3d.0} \quad d_{3d.2} = \begin{pmatrix} 2.75 \\ 3.75 \\ 12.06 \\ 14.69 \end{pmatrix} \cdot in \quad A_{3d.2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.12 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Area & Depth of Reinf. for: $w_{llr} = 35 \cdot psf$ & $w_{open_2} = 3\,ft$

given $a_{3d.2.g} := 2.71365\ in$ Initial Guess - Depth of Compression Block

$$f_{3d.2}\left(a_{3d.2.g}\right) := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2.g} + P \ldots$$
$$+ A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2.g} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{3d.2} := root\left(f_{3d.2}\left(a_{3d.2.g}\right), a_{3d.2.g}\right)$

$a_{3d.2} = 2.71365 \cdot in$

Depth of Compression Block

$c_{3d.2} := \frac{a_{3d.2}}{\beta_1}$ $c_{3d.2} = 3.289 \cdot in$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + P = -41.5 \cdot kip$

Concrete Compression Force

$$F1_{3d.2} := A_{3d.2_0} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_0}\right) \cdot E_s, f_y\right], -f_y\right] \quad F1_{3d.2} = 4 \cdot kip$$

$$F2_{3d.2} := A_{3d.2_1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_1}\right) \cdot E_s, f_y\right], -f_y\right] \quad F2_{3d.2} = 11.7 \cdot kip$$

$$F3_{3d.2} := A_{3d.2_2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_2}\right) \cdot E_s, f_y\right], -f_y\right] \quad F3_{3d.2} = 7.2 \cdot kip$$

$$F4_{3d.2} := A_{3d.2_3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{3d.2} \cdot \beta_1} \cdot d_{3d.2_3}\right) \cdot E_s, f_y\right], -f_y\right] \quad F4_{3d.2} = 18.6 \cdot kip$$

$-0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} + F1_{3d.2} + F2_{3d.2} + F3_{3d.2} + F4_{3d.2} = -0 \cdot kip$

$Mn_{3d.2} := -0.85 \cdot f'_c \cdot t_w \cdot a_{3d.2} \cdot \frac{a_{3d.2}}{2} + F1_{3d.2} \cdot d_{3d.2_0} + F2_{3d.2} \cdot d_{3d.2_1} \ldots + F3_{3d.2} \cdot d_{3d.2_2} + F4_{3d.2} \cdot d_{3d.2_3}$ $\quad Mn_{3d.2} = 29.9 \cdot kip \cdot ft$

Nominal Flexural Strength, Max Moment

$\varepsilon_{3d.2} := -0.003 + \frac{0.003}{c_{3d.2}} \cdot d_{3d.2_3}$ $\quad \varepsilon_{3d.2} = 0.0104$ $\quad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{3d.2} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{3d.2} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\quad \phi_{3d.2} = 0.9$ $\quad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{3d.2} := \phi_{3d.2} \cdot Mn_{3d.2}$ $\quad \phi Mn_{3d.2} = 26.9 \cdot k \cdot ft$ $\quad Mu_{ref} = \text{"Flexural Strength, Sec. (22.3)"}$

$\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.0}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3} = 0.43 \cdot in^2$

$\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) = 13.375 \cdot in$

$\rho_{3d.2} := \frac{\text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} > a_{3d.2}, A_{3d.2_2}, 0in^2\right)\right) + A_{3d.2_3}}{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2}, \text{mean}\left(d_{3d.2_2}, d_{3d.2_3}\right), d_{3d.2_3}\right)\right) \cdot t_w}$ $\quad \rho_{3d.2} = 0.008$

$\rho_{ref} = \text{"Reinf. Ratio"}$

$\rho'_{3d.2} := \sum \begin{pmatrix} \text{if}\left(d_{3d.2_0} = 0in, 0in^2, \text{if}\left(d_{3d.2_0} < a_{3d.1}, A_{3d.2_0}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_1} = 0in, 0in^2, \text{if}\left(d_{3d.2_1} < a_{3d.1}, A_{3d.2_1}, 0in^2\right)\right) \\ \text{if}\left(d_{3d.2_2} = 0in, 0in^2, \text{if}\left(d_{3d.2_2} < a_{3d.1}, A_{3d.2_2}, 0in^2\right)\right) \end{pmatrix} \cdot \left[\frac{1}{\left(t_w \cdot d_{3d.2_0}\right)}\right]$ $\quad \rho'_{3d.2} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$\lambda_{s.3d.2} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\text{if}\left(d_{3d.2_2} = 0in, d_{3d.2_3}, \text{if}\left(d_{3d.2_2} > a_{3d.2} \wedge a_{3d.2} < d_{3d.2_3}, d_{3d.2_2}, d_{3d.2_3}\right)\right)}{10 \cdot in}}}\right)$ $\quad \lambda_{s.3d.2} = 0.952$

$\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{3d.2.a} := 8 \cdot \lambda_{s.3d.2} \cdot \lambda \cdot \rho_{3d.2}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot t_w \cdot d_{3d.2_3}$ $\quad Vc_{3d.2.a} = 5.11 \cdot kip$

$Vc_{3d.2.b} = 16.75 \cdot kip$

$\phi Vc_{3d.2} := \phi_V \cdot \min\left(Vc_{3d.2.a}, Vc_{3d.2.b}\right)$ $\quad \phi Vc_{3d.2} = 3.83 \cdot kip$ $\quad \phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$Mu_{3d.2} := \frac{wu_{3d.2mx} \cdot \left(w_{open_2}\right)^2}{8}$ $\quad Mu_{3d.1} = 2.42 \cdot kip \cdot ft$ $\quad Vu_{3d.2} := \frac{\max\left(\left|wu_{3d.2mx}\right|, \left|wu_{3d.2mn}\right|\right) \cdot w_{open_2}}{2}$ $\quad Vu_{3d.2} = 1.78 \cdot kip$

$chk_{3d.9} := \text{if}\left(\phi Mn_{3d.2} \geq Mu_{3d.2}, \text{"OK"}, \text{"NG"}\right)$ $\quad chk_{3d.9} = \text{"OK"}$

$chk_{3d.10} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 \geq Vu_{3d.2} \\ \text{"Min. Stirrups Req'd"} & \text{if } \phi Vc_{3d.2} \cdot 0.5 < Vu_{3d.2} \wedge \phi Vc_{3d.2} \geq Vu_{3d.2} \\ \text{"Stirrups Req'd"} & \text{otherwise} \end{cases}$ $\quad chk_{3d.10} = \text{"OK"}$

$$\phi Vs_{3d.2} := \phi_V \cdot A_{s.3b} \cdot \frac{4in}{12in} \cdot \left(f_y \cdot \frac{h_{3d_2}}{2} \right) \qquad \phi Vs_{3d.2} = 1.27 \cdot kip$$ Shear Reinforcement Provided by Wire Mesh

$\phi Vn_{3d.2} := \phi Vc_{3d.2} + \phi Vs_{3d.2}$ $\phi Vn_{3d.2} = 5.108 \cdot kip$ ϕVc_{ref} = "Conc. Shear Strgth, Eqn (22.5.5.1)"

$chk_{3d.11} := if\left(chk_{3d.10} = "OK" , "OK" , if\left(chk_{3d.10} = "Stirrups Req'd" , if\left(\phi Vn_{3d.2} < 2Vu_{3d.2} , "NG" , "OK" \right), "OK" \right)\right)$

$chk_{3d.11}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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4) Floor

a. Floor Slab Design

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref}$ = "Floor Live Load Option 0"

$w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref}$ = "Floor Live Load Option 1"

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref}$ = "Floor Live Load Option 2"

$t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

WWF_{4a} = "WWF.4x4_w4xw4" Slab Reinforcement

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ X- Section Area, Slab Reinforcment

$s_{t.rib} = 2\,ft$ Trans. Floor Rib Spacing

$b_{4a} := 1ft$ Strip Width

$l_{4a} := s_{t.rib} - b_{w.4a}$ $l_{4a} = 1.67 \cdot ft$

$l_{4a.ref}$:= "Flr Slab Clear Span "

$D_{4a.mn} := \gamma_c \cdot t_{slab}$ $D_{4a.mn} = 19.67 \cdot psf$

$DL_{4a.mn.ref}$:= "Flr Slab Selftweight"

$D_{4a.mx} := D_{4a.mn} + w_{misc.f}$ $D_{4a.mx} = 23.67 \cdot psf$

$DL_{4a.mx.ref}$:= "Flr Slab DL, Incl. Misc."

Check Strength of Floor Slab

		Axial	Moment	Shear
Dead (+)	$D_{4a.mx} = 23.67 \cdot psf$	$P_{D.p.4a} := 0$	$M_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}^2}{8}$	$V_{D.p.4a} := \frac{D_{4a.mx} \cdot l_{4a}}{2}$
			$M_{D.p.4a} = 0.008 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.p.4a} = 0.02 \cdot \frac{kip}{ft}$
Dead (-)	$D_{4a.mn} = 19.67 \cdot psf$	$P_{D.n.4a} := 0$	$M_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}^2}{8}$	$V_{D.n.4a} := \frac{D_{4a.mn} \cdot l_{4a}}{2}$
			$M_{D.n.4a} = 0.007 \cdot \frac{kip \cdot ft}{ft}$	$V_{D.n.4a} = 0.016 \cdot \frac{kip}{ft}$
Live	$w_{llf.0} = 250 \cdot psf$	$P_{L.4a.1} := 0$	$M_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}^2}{8}$	$V_{L.4a.1} := \frac{w_{llf.0} \cdot l_{4a}}{2}$
			$M_{L.4a.1} = 0.087 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.1} = 0.208 \cdot \frac{kip}{ft}$
	$w_{llf.1} = 200 \cdot psf$	$P_{L.4a.2} := 0$	$M_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}^2}{8}$	$V_{L.4a.2} := \frac{w_{llf.1} \cdot l_{4a}}{2}$
			$M_{L.4a.2} = 0.069 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.2} = 0.167 \cdot \frac{kip}{ft}$
	$w_{llf.2} = 150 \cdot psf$	$P_{L.4a.3} := 0$	$M_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}^2}{8}$	$V_{L.4a.3} := \frac{w_{llf.2} \cdot l_{4a}}{2}$
			$M_{L.4a.3} = 0.052 \cdot \frac{kip \cdot ft}{ft}$	$V_{L.4a.3} = 0.125 \cdot \frac{kip}{ft}$
Fluid	$w_{F.4a} := 0$	$P_{F.4a} := 0$	$M_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}^2}{8}$	$V_{F.4a} := \frac{w_{F.4a} \cdot l_{4a}}{2}$
	$w_{F.4a} = 0 \cdot psf$		$M_{F.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{F.4a} = 0 \cdot \frac{kip}{ft}$
H	$w_{H.4a} := 0$	$P_{H.4a} := 0$	$M_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}^2}{8}$	$V_{H.4a} := \frac{w_{H.4a} \cdot l_{4a}}{2}$
	$w_{H.4a} = 0 \cdot psf$		$M_{H.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{H.4a} = 0 \cdot \frac{kip}{ft}$

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.co

Roof Live Load	$w_{RL.4a} := 0$	$P_{RL.4a} := 0$	$M_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}^2}{8}$	$V_{RL.4a} := \frac{w_{RL.4a} \cdot l_{4a}}{2}$
	$w_{RL.4a} = 0 \cdot psf$		$M_{RL.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{RL.4a} = 0 \cdot \frac{kip}{ft}$
Snow	$w_{S.4a} := 0$	$P_{S.4a} := 0$	$M_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}^2}{8}$	$V_{S.4a} := \frac{w_{S.4a} \cdot l_{4a}}{2}$
	$w_{S.4a} = 0 \cdot psf$		$M_{S.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{S.4a} = 0 \cdot \frac{kip}{ft}$
Rain	$w_{R.4a} := 0$	$P_{R.4a} := 0$	$M_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}^2}{8}$	$V_{R.4a} := \frac{w_{R.4a} \cdot l_{4a}}{2}$
	$w_{R.4a} = 0 \cdot psf$		$M_{R.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{R.4a} = 0 \cdot \frac{kip}{ft}$
Wind (+)	$w_{W.p.4a} := 0$	$P_{w.p.4a} := 0$	$M_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}^2}{8}$	$V_{w.p.4a} := \frac{w_{W.p.4a} \cdot l_{4a}}{2}$
	$w_{W.p.4a} = 0 \cdot psf$		$M_{w.p.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.p.4a} = 0 \cdot \frac{kip}{ft}$
Wind (-)	$w_{W.n.4a} := 0$	$P_{w.n.4a} := 0$	$M_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}^2}{8}$	$V_{w.n.4a} := \frac{w_{W.n.4a} \cdot l_{4a}}{2}$
	$w_{W.n.4a} = 0 \cdot psf$		$M_{w.n.4a} = 0 \cdot \frac{kip \cdot ft}{ft}$	$V_{w.n.4a} = 0 \cdot \frac{kip}{ft}$
Earthquake	$w_{E.4a} := 0.2 \cdot S_{DS} \cdot D_{4a.mx}$	$P_{E.4a} := 0$	$M_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}^2}{8}$	$V_{E.4a} := \frac{w_{E.4a} \cdot l_{4a}}{2}$
	$w_{E.4a} = 4.54 \cdot psf$		$M_{E.4a} = 0.002 \cdot \frac{kip \cdot ft}{ft}$	$V_{E.4a} = 0.004 \cdot \frac{kip}{ft}$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.1} = \begin{pmatrix} 0.012 \\ 0.149 \\ 0.097 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.098 \\ 0.098 \\ 0.006 \\ 0.006 \\ 0.008 \\ 0.008 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.1} = \begin{pmatrix} 0.028 \\ 0.357 \\ 0.232 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.236 \\ 0.236 \\ 0.015 \\ 0.015 \\ 0.019 \\ 0.019 \end{pmatrix} \cdot \frac{kip}{ft}$$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.2} = \begin{pmatrix} 0.012 \\ 0.121 \\ 0.079 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.081 \\ 0.081 \\ 0.006 \\ 0.006 \\ 0.008 \\ 0.008 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.2} = \begin{pmatrix} 0.028 \\ 0.29 \\ 0.19 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.194 \\ 0.194 \\ 0.015 \\ 0.015 \\ 0.019 \\ 0.019 \end{pmatrix} \cdot \frac{kip}{ft}$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4a.3} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad Mu_{4a.3} = \begin{pmatrix} 0.012 \\ 0.093 \\ 0.062 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.01 \\ 0.064 \\ 0.064 \\ 0.006 \\ 0.006 \\ 0.008 \\ 0.008 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft} \quad Vu_{4a.3} = \begin{pmatrix} 0.028 \\ 0.224 \\ 0.149 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.024 \\ 0.152 \\ 0.152 \\ 0.015 \\ 0.015 \\ 0.019 \\ 0.019 \end{pmatrix} \cdot \frac{kip}{ft}$$

$P_{u.4a.1.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.1.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mx} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.2.mn} = 0.0 \cdot \frac{kip}{ft}$ $P_{u.4a.3.mx} = 0.0 \cdot \frac{kip}{ft}$

$P_{u.4a.3.mn} = 0.0 \cdot \frac{kip}{ft}$ $Pu_{ref} := \text{"Ultimate Axial"}$

$M_{u.4a.1} = 0.15 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.2} = 0.12 \cdot \frac{kip \cdot ft}{ft}$ $M_{u.4a.3} = 0.09 \cdot \frac{kip \cdot ft}{ft}$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$V_{u.4a.1.mx} = 0.36 \cdot \frac{kip}{ft}$ $V_{u.4a.1.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mx} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.2.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.3.mx} = 0.22 \cdot \frac{kip}{ft}$

$V_{u.4a.3.mn} = 0.01 \cdot \frac{kip}{ft}$ $V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$ $Vu_{ref} = \text{"Ultimate Shear"}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$d_{4a} := \frac{t_{slab}}{2} - 0.25in$ $d_{4a} = 0.75 \cdot in$ $d_{ref} = \text{"Depth of Reinf."}$ $\rho_{4a} := \frac{A_{s.4a}}{d_{4a}}$ $\rho_{4a} = 0.0133$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$a_{4a} := \frac{A_{s.4a} \cdot f_y}{0.85 \cdot f'_c}$ $a_{4a} = 0.157 \cdot in$ $a_{ref} = \text{"Depth of Comp. Block, Eqn (22.2.2.4.1)"}$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$As_{mn.4a} := \rho_{t\_s.min} \cdot t_{slab} \cdot b_{4a}$ $As_{mn.4a} = 0.043 \cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$chk_{4a.1} := \text{if}(As_{mn.4a} \le A_{s.4a} \cdot b_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.1} = \text{"OK"}$

$\beta_1 = 0.825$ $\beta_{1.ref} = \text{"Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"}$

$c_{4a} := \frac{a_{4a}}{\beta_1}$ $c_{4a} = 0.19 \cdot in$ $\varepsilon_{4a} := -0.003 + \frac{0.003}{c_{4a}} \cdot d_{4a}$ $\varepsilon_{4a} = 0.0088$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$ $\varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$

$\phi_{4a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4a} = 0.9$ $\phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\phi Mn_{4a} := \phi_{4a} \cdot A_{s.4a} \cdot f_y \cdot \left(d_{4a} - \frac{a_{4a}}{2}\right)$ $\phi Mn_{4a} = 362.6 \cdot \frac{lb \cdot ft}{ft}$ $\phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$

$M_{u.4a.1} = 148.75 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.2} = 120.972 \cdot \frac{lb \cdot ft}{ft}$ $M_{u.4a.3} = 93.194 \cdot \frac{lb \cdot ft}{ft}$

$chk_{4a.2} := \text{if}(M_{u.4a.1} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.2} = \text{"OK"}$

$chk_{4a.3} := \text{if}(M_{u.4a.2} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.3} = \text{"OK"}$

$chk_{4a.4} := \text{if}(M_{u.4a.3} \le \phi Mn_{4a}, \text{"OK"}, \text{"NG"})$ $chk_{4a.4} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$\lambda_{s.4a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{4a}}{10 \cdot in}}}\right)$ $\lambda_{s.4a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4a.a.1} := 8 \cdot \lambda_{s.4a} \cdot \lambda \cdot \rho_{4a}^{\frac{1}{3}} \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.b.1} := 5 \cdot \lambda \cdot \left(\sqrt{f'_c \cdot psi} \cdot b_{4a} \cdot d_{4a}\right)$ $Vc_{4a.a.1} = 0.974 \cdot kip$ $Vc_{4a.b.1} = 2.566 \cdot kip$

$\phi Vc_{4a.min} := \phi_V \cdot \left(\frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}} + 8 \cdot \sqrt{f'_c \cdot psi} \cdot d_{4a}\right)$ $\phi Vc_{4a.min} = 4.353 \cdot \frac{kip}{ft}$ Geometry Max Shear Strength Eqn (22.5.1.2)

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

$chk_{4a.5} := \text{if}(V_{u.4a.1} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.6} := \text{if}(V_{u.4a.2} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$

$chk_{4a.7} := \text{if}(V_{u.4a.3} \le \phi Vc_{4a.min}, \text{"OK"}, \text{"NG"})$ $chk_{4a.5} = \text{"OK"}$ $chk_{4a.6} = \text{"OK"}$ $chk_{4a.7} = \text{"OK"}$

$\phi Vc_{4a} := \phi_V \cdot \frac{\min(Vc_{4a.a.1}, Vc_{4a.b.1})}{b_{4a}}$ $\phi Vc_{4a} = 0.73 \cdot \frac{kip}{ft}$ $\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$V_{u.4a.1} = 0.357 \cdot \frac{kip}{ft}$ $V_{u.4a.2} = 0.29 \cdot \frac{kip}{ft}$ $V_{u.4a.3} = 0.224 \cdot \frac{kip}{ft}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$chk_{4a.8} := if\left(2V_{u.4a.1} \le \phi Vc_{4a}, "OK", "NG"\right)$ $chk_{4a.9} := if\left(2V_{u.4a.2} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.10} := if\left(2V_{u.4a.3} \le \phi Vc_{4a}, "OK", "NG"\right)$

$chk_{4a.8} = "OK"$ $chk_{4a.9} = "OK"$ $chk_{4a.10} = "OK"$

Servicability

$f_r = 427.65 \cdot psi$ $f_{r.ref} = "Modulus of Rupture, Eqn (19.2.3.1)"$

$Ig_{4a} := \frac{1}{12} \cdot t_{slab}^{3} \cdot b_{4a}$ $Ig_{4a} = 8 \cdot in^4$

$I_{g.ref} = "Gross Moment of Inertia"$

$y_{t.4a} := \frac{t_{slab}}{2}$ $y_{t.4a} = 1 \cdot in$

$y_{t.ref} = "Dist., Centroid-Tension Face"$

$Mcr_{4a} := f_r \cdot \frac{Ig_{4a}}{y_{t.4a}}$ $Mcr_{4a} = 285.1 \cdot lb \cdot ft$ $M_{cr.ref} = "Crack Moment of Inertia, Eqn (24.2.3.5)"$

$M_{4a.1} := \frac{\left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.2} := \frac{\left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{4a.3} := \frac{\left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$

$M_{4a.1} = 95.02 \cdot lb \cdot ft$ $M_{4a.2} = 77.66 \cdot lb \cdot ft$ $M_{4a.3} = 60.3 \cdot lb \cdot ft$ $M_{S.ref} = "Service Moment"$

$chk_{4a.11} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.1}, "Cracks Unlikely", "Cracks"\right)$ $chk_{4a.12} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.2}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.13} := if\left(\frac{2}{3} Mcr_{4a} > M_{4a.3}, "Cracks Unlikely", "Cracks"\right)$

$chk_{4a.11} = "Cracks Unlikely"$ $chk_{4a.12} = "Cracks Unlikely"$ $chk_{4a.13} = "Cracks Unlikely"$

$M_{D.4a} := \frac{\left[\left(D_{4a.mx}\right) \cdot l_{4a}^{2}\right]}{8} \cdot b_{4a}$ $M_{D.4a} = 8.22 \cdot lb \cdot ft$ $M_{D.ref} = "Dead Load Moment"$

$0.5 \cdot Ig_{4a} = 4 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a} = -6.933 \cdot in^4$ $0.25 \cdot Ig_{4a} = 2 \cdot in^4$

$I_{cr.4a} := \min\left[0.5 \cdot Ig_{4a}, \max\left[0.25 \cdot Ig_{4a}, \left(0.1 + 25 \cdot \rho_{4a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{4a}}{d_{4a}}\right) \cdot Ig_{4a}\right]\right]$ $I_{cr.4a} = 2 \cdot in^4$

$I_{cr.slab.ref} = "Crack Mom of Inertia - Slab, Tbl (6.6.3.1.1a)"$

$I_{e.D.4a} := if\left[M_{D.4a} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{D.4a}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.D.4a} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.1} := if\left[M_{4a.1} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.1}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.1} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

$I_{e.S.4a.2} := if\left[M_{4a.2} < \frac{2}{3} Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3} Mcr_{4a}}{M_{4a.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right]$ $I_{e.S.4a.2} = 8 \cdot in^4$

$I_{e.ref} = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$I_{e.S.4a.3} := \text{if}\left[M_{4a.3} < \frac{2}{3}Mcr_{4a}, Ig_{4a}, \frac{I_{cr.4a}}{1 - \left(\frac{\frac{2}{3}Mcr_{4a}}{M_{4a.3}}\right)^2 \cdot \left(1 - \frac{I_{cr.4a}}{Ig_{4a}}\right)}\right] \qquad I_{e.S.4a.3} = 8 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{D.4a} := \frac{5 \cdot \left(D_{4a.mx} \cdot b_{4a}\right) \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.D.4a}} \qquad \Delta_{D.4a} = 0.00017 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.1}} \quad \Delta_{S.4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.2}} \quad \Delta_{S.4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot b_{4a}\right] \cdot l_{4a}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4a.3}}$$

$\Delta_{S.4a.1} = 0.00199 \cdot in$ $\Delta_{S.4a.2} = 0.00162 \cdot in$ $\Delta_{S.4a.3} = 0.00126 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{L.4a.1} := \Delta_{S.4a.1} - \Delta_{D.4a}$ $\Delta_{L.4a.2} := \Delta_{S.4a.2} - \Delta_{D.4a}$ $\Delta_{L.4a.3} := \Delta_{S.4a.3} - \Delta_{D.4a}$

$\Delta_{L.4a.1} = 0.00181 \cdot in$ $\Delta_{L.4a.2} = 0.00145 \cdot in$ $\Delta_{L.4a.3} = 0.00109 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4a} := 0$ $\rho'_{4a} = 0$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4a.1} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.1} = 0.0041 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.2} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.2} = 0.0034 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{LT.4a.3} := \Delta_{D.4a} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4a}}\right) + \Delta_{L.4a.3} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4a}}\right) \qquad \Delta_{LT.4a.3} = 0.0027 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta g_{4a.1} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.0}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.2} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.1}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a} \quad \Delta g_{4a.3} := \frac{5 \cdot \left[\left(D_{4a.mx} + w_{llf.2}\right) \cdot l_{4a}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4a}} \cdot b_{4a}$$

$\Delta g_{4a.1} = 0.002 \cdot in$ $\Delta g_{4a.2} = 0.0016 \cdot in$ $\Delta g_{4a.3} = 0.0013 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4a} := \frac{l_{4a}}{\eta_{a.RF.NonDam}} \qquad \delta_{4a} = 0.0833 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$\Delta_{4a.1} := \Delta g_{4a.1} + \Delta_{LT.4a.1}$ $\Delta_{4a.2} := \Delta g_{4a.2} + \Delta_{LT.4a.2}$ $\Delta_{4a.3} := \Delta g_{4a.3} + \Delta_{LT.4a.3}$

$\Delta_{4a.1} = 0.0061 \cdot in$ $\Delta_{4a.2} = 0.005 \cdot in$ $\Delta_{4a.3} = 0.004 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4a.1} := \frac{l_{4a}}{\Delta_{4a.1}}$ $\eta_{4a.1} = 3263.5$ $\eta_{4a.2} := \frac{l_{4a}}{\Delta_{4a.2}}$ $\eta_{4a.2} = 3968.1$ $\eta_{4a.3} := \frac{l_{4a}}{\Delta_{4a.3}}$ $\eta_{4a.3} = 5061$ η_{ref} = "Deflection Ratio"

$chk_{4a.14} := \text{if}\left(\Delta_{4a.1} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.15} := \text{if}\left(\Delta_{4a.2} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4a.16} := \text{if}\left(\Delta_{4a.3} < \delta_{4a}, \text{"OK"}, \text{"NG"}\right)$

$chk_{4a.14}$ = "OK" $chk_{4a.15}$ = "OK" $chk_{4a.16}$ = "OK"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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b. Floor Rib Design

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $b_{w.4a} = 4 \cdot in$ $b_{w.4a.ref}$ = "Width of Floor Rib, Trans."

$W_f = 10.958\ ft$ $W_{f.ref}$ = "Width of Floor Panel" $d_{rib} = 6 \cdot in$ $d_{rib.ref}$ = "Depth of Floor Rib"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam"

$l_{4b} = 112 \cdot in$ $l_{4b.ref}$ = "Clear Span, Dist Btwn SW Beam" $l_{4a} = 20 \cdot in$ $l_{4a.ref}$ = "Flr Slab Clear Span "

$CC_4 := 0.75in$ Clear Cover

$D_{4a.mx} = 23.7 \cdot psf$ $DL_{4a.mx.ref}$ = "Flr Slab DL, Incl. Misc." $D_{4a.mn} = 19.7 \cdot psf$ $DL_{4a.mn.ref}$ = "Flr Slab Selftweight"

$wdl_{4b.mn} := \gamma_c \cdot b_{w.4a} \cdot d_{rib} + D_{4a.mn} \cdot s_{t.rib}$ $wdl_{4b.mn} = 59 \cdot plf$ $wdl_{4b.mn.ref}$:= "Min. Flr Rib Selfweight, Incl. Flange"

$wdl_{4b.mx} := wdl_{4b.mn} + w_{misc.r} \cdot s_{t.rib}$ $wdl_{4b.mx} = 73 \cdot plf$ $wdl_{4b.mx.ref}$:= "Max. Flr Rib Selfweight, Incl. Flange"

$wll_{4b.0} := w_{llf.0} \cdot s_{t.rib}$ $wll_{4b.0} = 500 \cdot plf$ $wll_{4b.0.ref}$:= "LL Opt. 0 On Flr Rib"

$wll_{4b.1} := w_{llf.1} \cdot s_{t.rib}$ $wll_{4b.1} = 400 \cdot plf$ $wll_{4b.1.ref}$:= "LL Opt. 1 On Flr Rib"

$wll_{4b.2} := w_{llf.2} \cdot s_{t.rib}$ $wll_{4b.2} = 300 \cdot plf$ $wll_{4b.2.ref}$:= "LL Opt. 2 On Flr Rib"

Check Strength of Floor Rib

		Axial	Moment	Shear
Dead (+)	$wdl_{4b.mx} = 73 \cdot plf$	$P_{D.p.4b} := 0$	$M_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$	$V_{D.p.4b} := \frac{wdl_{4b.mx} \cdot l_{4b}}{2}$
			$M_{D.p.4b} = 0.795 \cdot kip \cdot ft$	$V_{D.p.4b} = 0.341 \cdot kip$
Dead (-)	$wdl_{4b.mn} = 59 \cdot plf$	$P_{D.n.4b} := 0$	$M_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}^2}{8}$	$V_{D.n.4b} := \frac{wdl_{4b.mn} \cdot l_{4b}}{2}$
			$M_{D.n.4b} = 0.642 \cdot kip \cdot ft$	$V_{D.n.4b} = 0.275 \cdot kip$
Live	$wll_{4b.0} = 500 \cdot plf$	$P_{L.4b.1} := 0$	$M_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}^2}{8}$	$V_{L.4b.0} := \frac{wll_{4b.0} \cdot l_{4b}}{2}$
			$M_{L.4b.0} = 5.444 \cdot kip \cdot ft$	$V_{L.4b.0} = 2.333 \cdot kip$
	$wll_{4b.1} = 400 \cdot plf$	$P_{L.4b.2} := 0$	$M_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}^2}{8}$	$V_{L.4b.1} := \frac{wll_{4b.1} \cdot l_{4b}}{2}$
			$M_{L.4b.1} = 4.356 \cdot kip \cdot ft$	$V_{L.4b.1} = 1.867 \cdot kip$
	$wll_{4b.2} = 300 \cdot plf$	$P_{L.4b.3} := 0$	$M_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}^2}{8}$	$V_{L.4b.2} := \frac{wll_{4b.2} \cdot l_{4b}}{2}$
			$M_{L.4b.2} = 3.267 \cdot kip \cdot ft$	$V_{L.4b.2} = 1.4 \cdot kip$
Fluid	$w_{F.4b} := 0$	$P_{F.4b} := 0$	$M_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}^2}{8}$	$V_{F.4b} := \frac{w_{F.4b} \cdot l_{4b}}{2}$
			$M_{F.4b} = 0 \cdot kip \cdot ft$	$V_{F.4b} = 0 \cdot kip$
H	$w_{H.4b} := 0$	$P_{H.4b} := 0$	$M_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}^2}{8}$	$V_{H.4b} := \frac{w_{H.4b} \cdot l_{4b}}{2}$
			$M_{H.4b} = 0 \cdot kip \cdot ft$	$V_{H.4b} = 0 \cdot kip$

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Roof Live Load	$w_{RL.4b} := 0$	$P_{Lr.4b} := 0$	$M_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}^2}{8}$	$V_{RL.4b} := \frac{w_{RL.4b} \cdot l_{4b}}{2}$
			$M_{RL.4b} = 0 \cdot kip \cdot ft$	$V_{RL.4b} = 0 \cdot kip$
Snow	$w_{S.4b} := 0$	$P_{S.4b} := 0$	$M_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}^2}{8}$	$V_{S.4b} := \frac{w_{S.4b} \cdot l_{4b}}{2}$
			$M_{S.4b} = 0 \cdot kip \cdot ft$	$V_{S.4b} = 0 \cdot kip$
Rain	$w_{R.4b} := 0$	$P_{R.4b} := 0$	$M_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}^2}{8}$	$V_{R.4b} := \frac{w_{R.4b} \cdot l_{4b}}{2}$
			$M_{R.4b} = 0 \cdot kip \cdot ft$	$V_{R.4b} = 0 \cdot kip$
Wind (+)	$w_{W.p.4b} := 0$	$P_{w.p.4b} := 0$	$M_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}^2}{8}$	$V_{w.p.4b} := \frac{w_{W.p.4b} \cdot l_{4b}}{2}$
			$M_{w.p.4b} = 0 \cdot kip \cdot ft$	$V_{w.p.4b} = 0 \cdot kip$
Wind (-)	$w_{W.n.4b} := 0$	$P_{w.n.4b} := 0$	$M_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}^2}{8}$	$V_{w.n.4b} := \frac{w_{W.n.4b} \cdot l_{4b}}{2}$
			$M_{w.n.4b} = 0 \cdot kip \cdot ft$	$V_{w.n.4b} = 0 \cdot kip$
Earthquake	$w_{E.4b} := 0.2 \cdot S_{DS} \cdot wdl_{4b.m}$	$P_{E.4b} := 0$	$M_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}^2}{8}$	$V_{E.4b} := \frac{w_{E.4b} \cdot l_{4b}}{2}$
	$w_{E.4b} = 14.02 \cdot plf$		$M_{E.4b} = 0.153 \cdot kip \cdot ft$	$V_{E.4b} = 0.065 \cdot kip$

Factored Loads, Floor Live Load Option 1, $w_{llf.0} = 250 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.0} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.0} = \begin{pmatrix} 1.11 \\ 9.66 \\ 6.4 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 6.55 \\ 6.55 \\ 0.58 \\ 0.58 \\ 0.73 \\ 0.73 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.0} = \begin{pmatrix} 0.48 \\ 4.14 \\ 2.74 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.81 \\ 2.81 \\ 0.25 \\ 0.25 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot kip$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Factored Loads, Floor Live Load Option 2, $w_{llf.1} = 200 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.1} = \begin{pmatrix} 1.11 \\ 7.92 \\ 5.31 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 5.46 \\ 5.46 \\ 0.58 \\ 0.58 \\ 0.73 \\ 0.73 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.1} = \begin{pmatrix} 0.48 \\ 3.4 \\ 2.28 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 2.34 \\ 2.34 \\ 0.25 \\ 0.25 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot kip$$

Factored Loads, Floor Live Load Option 3, $w_{llf.2} = 150 \cdot psf$

$$LRFD^{\langle 1 \rangle} = \begin{pmatrix} \text{"1.4(D+F)"} \\ \text{"1.2(D+F)+1.6(L+H)+0.5(max(Lr,S,R)) "} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+1.0(L)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W+)"} \\ \text{"1.2(D+F)+1.6(max(Lr,S,R)+H)+0.5(W-)"} \\ \text{"1.2(D+F)+1.0(W+)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(W-)+1.0(H)+0.5(max(Lr,S,R))"} \\ \text{"1.2(D+F)+1.0(E+)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"1.2(D+F)+1.0(E-)+1.0(L)+1.6(H)+0.2(S)"} \\ \text{"0.9(D+F)+1.0(W+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(W-)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E+)+1.6(H)"} \\ \text{"0.9(D+F)+1.0(E-)+1.6(H)"} \end{pmatrix} \quad Pu_{4b.2} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot lbf \quad Mu_{4b.2} = \begin{pmatrix} 1.11 \\ 6.18 \\ 4.22 \\ 0.95 \\ 0.95 \\ 0.95 \\ 0.95 \\ 4.37 \\ 4.37 \\ 0.58 \\ 0.58 \\ 0.73 \\ 0.73 \end{pmatrix} \cdot kip \cdot ft \quad Vu_{4b.2} = \begin{pmatrix} 0.48 \\ 2.65 \\ 1.81 \\ 0.41 \\ 0.41 \\ 0.41 \\ 0.41 \\ 1.87 \\ 1.87 \\ 0.25 \\ 0.25 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot kip$$

$P_{u.4b.0.mx} = 0.0 \cdot kip$ $P_{u.4b.0.mn} = 0.0 \cdot kip$ $P_{u.4b.1.mx} = 0.0 \cdot kip$ $P_{u.4b.1.mn} = 0.0 \cdot kip$ $P_{u.4b.2.mx} = 0.0 \cdot kip$

$P_{u.4b.2.mn} = 0.0 \cdot kip$ $Pu_{ref} = \text{"Ultimate Axial"}$

$M_{u.4b.0a} = 9.66 \cdot kip \cdot ft$ $M_{u.4b.1a} = 7.92 \cdot kip \cdot ft$ $M_{u.4b.2a} = 6.18 \cdot kip \cdot ft$ $Mu_{p.ref} = \text{"Ultimate Moment, +ve"}$

$M_{u.4b.0b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.58 \cdot kip \cdot ft$ $M_{u.4b.2b} = 0.58 \cdot kip \cdot ft$ $Mu_{n.ref} = \text{"Ultimate Moment, -ve"}$

$V_{u.mx.4b.0} = 4.14 \cdot kip$ $V_{u.mn.4b.0} = 0.25 \cdot kip$ $V_{u.mx.4b.1} = 3.4 \cdot kip$ $V_{u.mn.4b.1} = 0.25 \cdot kip$ $V_{u.mx.4b.2} = 2.65 \cdot kip$

$V_{u.mn.4b.2} = 0.25 \cdot kip$ $V_{u.4b.0} = 4.142 \cdot kip$ $V_{u.4b.1} = 3.395 \cdot kip$ $V_{u.4b.2} = 2.649 \cdot kip$ $Vu_{ref} = \text{"Ultimate Shear"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Reinforcement, Floor Live Load Option 0, $w_{llf.0} = 250 \cdot psf$

$B_{4b.a.0} = "\#7"$ $B_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.a.0} = 0.875 \cdot in$ $D_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.a.0} = 1$ $N_{4b.a.0.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.a.0} = 0.6 \cdot in^2$ $A_{4b.a.0.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.b.0} = "\#4"$ $B_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 0"$ $D_{4b.b.0} = 0.5 \cdot in$ $D_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.b.0} = 1$ $N_{4b.b.0.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 0"$ $A_{4b.b.0} = 0.2 \cdot in^2$ $A_{4b.b.0.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 0"$

$B_{4b.c.0} = "\#3"$ $B_{4b.c.0.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 0"$ $D_{4b.c.0} = 0.375 \cdot in$ $D_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 0"$

$N_{4b.c.0} = 1$ $N_{4b.c.0.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 0"$ $A_{4b.c.0} = 0.11 \cdot in^2$ $A_{4b.c.0.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 0"$

Maximum Flexure

$h_{cg.4b.0} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a0} := t_{floor} - h_{cg.4b.0} - CC_4 + D_{4b.c.0} + \frac{D_{4b.b.0}}{2}$ $A1_{4b.a0} := A_{4b.b.0}$ $d1_{4b.a0} = 1.625 \cdot in$ $A1_{4b.a0} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a0} := 0in$ $A2_{4b.a0} := 0in^2$ $d2_{4b.a0} = 0 \cdot in$ $A2_{4b.a0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a0} := t_{floor} - CC_4 - D_{4b.c.0} - \frac{D_{4b.a.0}}{2}$ $A3_{4b.a0} := A_{4b.a.0}$ $d3_{4b.a0} = 6.437 \cdot in$ $A3_{4b.a0} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b0} := CC_4 + D_{4b.c.0} + \frac{D_{4b.a.0}}{2}$ $A1_{4b.b0} := A3_{4b.a0}$ $d1_{4b.b0} = 1.562 \cdot in$ $A1_{4b.b0} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b0} := 0in$ $A2_{4b.b0} := A2_{4b.a0}$ $d2_{4b.b0} = 0 \cdot in$ $A2_{4b.b0} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b0} := CC_4 + h_{cg.4b.0} - D_{4b.c.0} - \frac{D_{4b.b.0}}{2}$ $A3_{4b.b0} := A1_{4b.a0}$ $d3_{4b.b0} = 6.375 \cdot in$ $A3_{4b.b0} = 0.2 \cdot in^2$ Depth/Area Layer 2

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv.\ Rect.\ Conc\ Stress\ Distrib.,\ Tbl\ (22.2.2.4.3)"$

given $ag_{4b} := 2.22324\ in$ Initial Guess - Depth of Compression Block

Reinforcement, Floor Live Load Option 1, $w_{llf.1} = 200 \cdot psf$

$B_{4b.a.1} = "\#7"$ $B_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.a.1} = 0.875 \cdot in$ $D_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.a.1} = 1$ $N_{4b.a.1.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.a.1} = 0.6 \cdot in^2$ $A_{4b.a.1.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.b.1} = "\#4"$ $B_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 1"$ $D_{4b.b.1} = 0.5 \cdot in$ $D_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.b.1} = 1$ $N_{4b.b.1.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 1"$ $A_{4b.b.1} = 0.2 \cdot in^2$ $A_{4b.b.1.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 1"$

$B_{4b.c.1} = "\#3"$ $B_{4b.c.1.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 1"$ $D_{4b.c.1} = 0.375 \cdot in$ $D_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 1"$

$N_{4b.c.1} = 1$ $N_{4b.c.1.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 1"$ $A_{4b.c.1} = 0.11 \cdot in^2$ $A_{4b.c.1.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 1"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

Maximum Flexure

$h_{cg.4b.1} := 6.25in$ Total Height of Cage

Positive Flexure

$d1_{4b.a1} := t_{floor} - h_{cg.4b.1} - CC_4 + D_{4b.c.1} + \frac{D_{4b.b.1}}{2}$ $A1_{4b.a1} := A_{4b.b.1}$ $d1_{4b.a1} = 1.625 \cdot in$ $A1_{4b.a1} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a1} := 0in$ $A2_{4b.a2} := 0in^2$ $d2_{4b.a1} = 0 \cdot in$ $A2_{4b.a2} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a1} := t_{floor} - CC_4 - D_{4b.c.1} - \frac{D_{4b.a.1}}{2}$ $A3_{4b.a1} := A_{4b.a.1}$ $d3_{4b.a1} = 6.437 \cdot in$ $A3_{4b.a1} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b1} := CC_4 + D_{4b.c.1} + \frac{D_{4b.a.1}}{2}$ $A1_{4b.b1} := A3_{4b.a1}$ $d1_{4b.b1} = 1.562 \cdot in$ $A1_{4b.b1} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b1} := 0in$ $A2_{4b.b1} := A2_{4b.a2}$ $d2_{4b.b1} = 0 \cdot in$ $A2_{4b.b1} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.b1} := CC_4 + h_{cg.4b.1} - D_{4b.c.1} - \frac{D_{4b.b.1}}{2}$ $A3_{4b.b1} := A1_{4b.a1}$ $d3_{4b.b1} = 6.375 \cdot in$ $A3_{4b.b1} = 0.2 \cdot in^2$ Depth/Area Layer 2

Reinforcement, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$B_{4b.a.2} = "\#7"$ $B_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.a.2} = 0.875 \cdot in$ $D_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.a.2} = 1$ $N_{4b.a.2.ref} = "\ \#\ Bot.\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.a.2} = 0.6 \cdot in^2$ $A_{4b.a.2.ref} = "Flr\ Rib\ Bot.\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.b.2} = "\#4"$ $B_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar,\ LL\ Opt.\ 2"$ $D_{4b.b.2} = 0.5 \cdot in$ $D_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.b.2} = 1$ $N_{4b.b.2.ref} = "\ \#\ Top\ Bar,\ LL\ Opt.\ 2"$ $A_{4b.b.2} = 0.2 \cdot in^2$ $A_{4b.b.2.ref} = "Flr\ Rib\ Top\ Bar\ Area,\ LL\ Opt.\ 2"$

$B_{4b.c.2} = "\#3"$ $B_{4b.c.2.ref} = "Flr\ Rib\ Stirrup,\ LL\ Opt.\ 2"$ $D_{4b.c.2} = 0.375 \cdot in$ $D_{4b.c.2.ref} = "Flr\ Rib\ Stirrup\ Diam.,\ LL\ Opt.\ 2"$

$N_{4b.c.2} = 1$ $N_{4b.c.2.ref} = "\ \#\ Stirrup\ Legs,\ LL\ Opt.\ 2"$ $A_{4b.c.2} = 0.11 \cdot in^2$ $A_{4b.c2.ref} = "Flr\ Rib\ Stirrup\ Area,\ LL\ Opt.\ 2"$

Maximum Flexure

$h_{cg.4b.2} := 6.25in$ Total Height of Cage

Positve Flexure

$d1_{4b.a2} := t_{floor} - h_{cg.4b.2} - CC_4 + D_{4b.c.2} + \frac{D_{4b.b.2}}{2}$ $A1_{4b.a2} := A_{4b.b.2}$ $d1_{4b.a2} = 1.625 \cdot in$ $A1_{4b.a2} = 0.2 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.a2} := 0in$ $A2_{4b.a3} := 0in^2$ $d2_{4b.a2} = 0 \cdot in$ $A2_{4b.a3} = 0 \cdot in^2$ Depth/Area Layer 2

$d3_{4b.a2} := t_{floor} - CC_4 - D_{4b.c.2} - \frac{D_{4b.a.2}}{2}$ $A3_{4b.a2} := A_{4b.a.2}$ $d3_{4b.a2} = 6.437 \cdot in$ $A3_{4b.a2} = 0.6 \cdot in^2$ Depth/Area Layer 2

Negative Flexure

$d1_{4b.b2} := CC_4 + D_{4b.c.2} + \frac{D_{4b.a.2}}{2}$ $A1_{4b.b2} := A3_{4b.a2}$ $d1_{4b.b2} = 1.562 \cdot in$ $A1_{4b.b2} = 0.6 \cdot in^2$ Depth/Area Layer 1

$d2_{4b.b2} := 0in$ $A2_{4b.b2} := A2_{4b.a3}$ $d2_{4b.b2} = 0 \cdot in$ $A2_{4b.b2} = 0 \cdot in^2$ Depth/Area Layer 2

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$d3_{4b.b2} := CC_4 + h_{cg.4b.2} - D_{4b.c.2} - \frac{D_{4b.b.2}}{2} \qquad A3_{4b.b2} := A1_{4b.a2} \qquad d3_{4b.b2} = 6.375\cdot in \quad A3_{4b.b2} = 0.2\cdot in^2$$

Depth/Area Layer 2

Strength, Floor Live Load Option 0, $w_{llf.0} = 250\cdot psf$

$$f_{4b.0a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0a} := \text{root}\left(f_{4b.0a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0a} = 2.22324\cdot in$

Depth of Compression Block

$c_{4b.0a} := \frac{a_{4b.0a}}{\beta_1} \qquad c_{4b.0a} = 2.695\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} = -34.016\cdot kip$

Concrete Compression Force

$$F1_{4b.0a} := A1_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d1_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0a} = -1.984\cdot kip$$

$$F2_{4b.0a} := A2_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d2_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0a} = 0\cdot kip$$

$$F3_{4b.0a} := A3_{4b.a0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0a}\cdot\beta_1}\cdot d3_{4b.a0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0a} + F1_{4b.0a} + F2_{4b.0a} + F3_{4b.0a} = 0\cdot kip$

$$Mn_{4b.0a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0a}^2}{2} + F1_{4b.0a}\cdot d1_{4b.a0} + F2_{4b.0a}\cdot d2_{4b.a0} + F3_{4b.0a}\cdot d3_{4b.a0} \qquad Mn_{4b.0a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.0b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.0.mn} \ \cdots$$
$$+ A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \ \cdots$$
$$+ A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.0b} := \text{root}\left(f_{4b.0b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.0b} = 1.54766\cdot in$

Depth of Compression Block

$c_{4b.0b} := \frac{a_{4b.0b}}{\beta_1} \qquad c_{4b.0b} = 1.876\cdot in$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} = -23.679\cdot kip$

Concrete Compression Force

$$F1_{4b.0b} := A1_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d1_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.0b} = 11.679\cdot kip$$

$$F2_{4b.0b} := A2_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d2_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.0b} = 0\cdot kip$$

$$F3_{4b.0b} := A3_{4b.b0}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.0b}\cdot\beta_1}\cdot d3_{4b.b0}\right)\cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.0b} = 12\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.0b} + F1_{4b.0b} + F2_{4b.0b} + F3_{4b.0b} = 0\cdot kip$

$$Mn_{4b.0b} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.0b}^2}{2} + F1_{4b.0b}\cdot d1_{4b.b0} + F2_{4b.0b}\cdot d2_{4b.b0} + F3_{4b.0b}\cdot d3_{4b.b0} \qquad Mn_{4b.0b} = 6.369\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.0a} := -0.003 + \frac{0.003}{c_{4b.0a}}\cdot d3_{4b.a0} \qquad \varepsilon_{4b.0a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$\varepsilon_{4b.0b} := -0.003 + \frac{0.003}{c_{4b.0b}} \cdot d3_{4b.b0}$ $\varepsilon_{4b.0b} = 0.0072$ $\varepsilon_{ref} =$ "Tensile Strain, Extreme Layer"

$\phi_{4b.0a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0a} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0a} = 0.831$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi_{4b.0b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.0b} - 0.002\right)\cdot\left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{4b.0b} = 0.9$ $\phi_{ref} =$ "Strgth Reduc. Fact, Tbl (21.2.1)"

$\phi Mn_{4b.0a} := \phi_{4b.0a}\cdot A3_{4b.a0}\cdot f_y\cdot\left(d3_{4b.a0} - \frac{a_{4b.0a}}{2}\right)$ $\phi Mn_{4b.0a} = 13.27\cdot k\cdot ft$ $\phi M_{p.ref} =$ "Flexural Strength, +ve, Sect (22.3)"

$\phi Mn_{4b.0b} := \phi_{4b.0b}\cdot A3_{4b.b0}\cdot f_y\cdot\left(d3_{4b.b0} - \frac{a_{4b.0b}}{2}\right)$ $\phi Mn_{4b.0b} = 5.04\cdot k\cdot ft$ $\phi M_{n.ref} =$ "Flexural Strength, -ve, Sect (22.3)"

$\rho_{4b.0a} := \frac{A3_{4b.a0}}{d3_{4b.a0}\cdot b_{w.4a}}$ $\rho_{4b.0a} = 0.0233$ $\rho_{4b.0b} := \frac{A3_{4b.b0}}{d3_{4b.b0}\cdot b_{w.4a}}$ $\rho_{4b.0b} = 0.0078$ $\rho_{ref} =$ "Reinf. Ratio"

$$\rho'_{4b.0a} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.a0} = 0in, 0in^2, \text{if}\left(d1_{4b.a0} < a_{4b.0a}, A1_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.a0} = 0in, 0in^2, \text{if}\left(d2_{4b.a0} < a_{4b.0a}, A2_{4b.a0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.a0} = 0in, 0in^2, \text{if}\left(d3_{4b.a0} < a_{4b.0a}, A3_{4b.a0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.a0}}$$

$\rho'_{4b.0a} = 0.0308$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$$\rho'_{4b.0b} := \frac{\sum\begin{pmatrix}\text{if}\left(d1_{4b.b0} = 0in, 0in^2, \text{if}\left(d1_{4b.b0} < a_{4b.0b}, A1_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d2_{4b.b0} = 0in, 0in^2, \text{if}\left(d2_{4b.b0} < a_{4b.0b}, A2_{4b.b0}, 0in^2\right)\right)\\ \text{if}\left(d3_{4b.b0} = 0in, 0in^2, \text{if}\left(d3_{4b.b0} < a_{4b.0b}, A3_{4b.b0}, 0in^2\right)\right)\end{pmatrix}}{b_{w.4a}\cdot d1_{4b.b0}}$$

$\rho'_{4b.0b} = 0$

$\rho'_{ref} =$ "Comp. Reinf. Ratio"

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} =$ "Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"

$A_{mn.4b.0} := \max\left(a1_{min}, a2_{min}\right)\cdot b_{w.4a}\cdot\max\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $A_{mn.4b.0} = 0.086\cdot in^2$

$chk_{4b.1} := \text{if}\left(A_{mn.4b.0} < \min\left(A1_{4b.a0}, A1_{4b.b0}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.1} =$ "OK"

$\phi_V = 0.75$ $\phi_{V.ref} =$ "Shear Strgth Reduc. Fact, Tbl (21.2.1)"

$\lambda = 0.85$ $\lambda_{ref} =$ "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"

$\lambda_{s.4b.0a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a0}, d3_{4b.b0}\right)}{10\cdot in}}}\right)$ $\lambda_{s.4b.0a} = 1$ $\lambda_{s.ref} =$ "Size Effect Fact, Eqn (22.5.5.1.3)"

$Vc_{4b.0.a} := 8\cdot\lambda_{s.4b.0a}\cdot\lambda\cdot\rho_{4b.0a}^{\frac{1}{3}}\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$ $Vc_{4b.0.b} := 5\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot b_{w.4a}\cdot\min\left(d3_{4b.a0}, d3_{4b.b0}\right)$

$Vc_{4b.0.a} = 3.32\cdot kip$ $Vc_{4b.0.b} = 7.27\cdot kip$ $\phi Vc_{4b.0} := \phi_V\cdot\min\left(Vc_{4b.0.a}, Vc_{4b.0.b}\right)$ $\phi Vc_{4b.0} = 2.492\cdot kip$

$\phi Vc_{ref} =$ "Conc. Shear Strgth, Eqn (22.5.5.1)"

$M_{u.4b.0a} = 9.665\cdot kip\cdot ft$ $\phi Mn_{4b.0a} = 13.27\cdot kip\cdot ft$ $M_{u.4b.0b} = 0.578\cdot kip\cdot ft$ $\phi Mn_{4b.0b} = 5.041\cdot kip\cdot ft$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$chk_{4b.2} := if(\phi Mn_{4b.0a} \geq M_{u.4b.0a}, "OK", "NG")$ $chk_{4b.3} := if(\phi Mn_{4b.0b} \geq M_{u.4b.0b}, "OK", "NG")$ $chk_{4b.2} = "OK"$ $chk_{4b.3} = "OK"$

$V_{u.4b.0} = 4.142 \cdot kip$ $2V_{u.4b.0} = 8.284 \cdot kip$ $\phi Vc_{4b.0} = 2.492 \cdot kip$

$chk_{4b.1c} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq V_{u.4b.0} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < V_{u.4b.0} \wedge \phi Vc_{4b.0} \geq V_{u.4b.0} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$ $chk_{4b.1c} = "Stirrups\ Required"$

$S_{4b.0} := floor\left(\frac{\min(d3_{4b.a0}, d3_{4b.b0})}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.0} := \frac{A_{4b.c.0} \cdot f_y \cdot \min(d3_{4b.a0}, d3_{4b.b0})}{S_{4b.0}}$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.4} := if\left(V_{s.4b.0} \geq \frac{V_{u.4b.0}}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4b.4} = "OK"$

$x_{stir.4b.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.0} - \max(Vu_{4b.0})}{-\max(|w_{u.mx.4b.0}|, |w_{u.mn.4b.0}|)}\right)\right)$

$x_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$l_{stir.4b.0} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.0}\right) + S_{4b.0}\right] \cdot 2 \leq 0 \\ x_{stir.4b.0} & \text{otherwise} \end{cases}$

$l_{stir.4b.0} = 22.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.0} := if\left(l_{stir.4b.0} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4b.0}}{S_{4b.0}} + 1\right)\right)$

$No_{stir.4b.0} = 9$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

L Beam Properties - $Beam_{4b.0.L} := "L\text{-}Beam"$

$b_{e1.L.4b.0} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.0} = 13.33 \cdot in$ $b_{e2.L.4b.0} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.0} = 16 \cdot in$

$b_{e3.L.4b.0} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.0} = 14 \cdot in$ $b_{ef.L.4b.0} := \min(b_{e1.L.4b.0}, b_{e2.L.4b.0}, b_{e3.L.4b.0})$ $b_{ef.L.4b.0} = 13.33 \cdot in$

$y_{t.L4.0} := \frac{b_{4a} \cdot (t_{floor} - t_{slab}) \cdot \frac{(t_{floor} - t_{slab})}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot (t_{floor} - t_{slab}) + t_{slab} \cdot b_{w.4a}}$ $y_{t.L4.0} = 5.033 \cdot in$

$y_{t.ref} = "Dist.,\ Centroid\text{-}Tension\ Face"$

$Ig_{L.4b.0} := \frac{1}{12} \cdot b_{w.4a} \cdot (t_{floor} - t_{slab})^3 + b_{w.4a} \cdot (t_{floor} - t_{slab}) \cdot \left[y_{t.L4.0} - \frac{(t_{floor} - t_{slab})}{2}\right]^2 \ldots$

$+ \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.0} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$

$Ig_{L.4b.0} = 283.3 \cdot in^4$

$I_{g.ref} = "Gross\ Moment\ of\ Inertia"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
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Rectangular Beam Properties - $Beam_{4b0.R} := \text{"Rectangular"}$

$b_{ef.R.4b.0} := b_{w.4a}$ $b_{ef.R.4b.0} = 4\cdot in$ $y_{t.R4.0} := \frac{t_{floor}}{2}$ $y_{t.R4.0} = 4\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.0} := \frac{1}{12}\cdot b_{ef.R.4b.0}\cdot t_{floor}^{3}$ $Ig_{R.4b.0} = 170.7\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.0} := Beam_{4b0.L}$ For Deflection

$b_{ef.4b.0} = 13.333\cdot in$ Effective Flange Width $Ig_{4b.0} = 283.3\cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.0} = 5.033\cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.0} := f_r\cdot\frac{Ig_{4b.0}}{y_{t.4b.0}}$ $Mcr_{4b.0} = 2.01\cdot kip\cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.0} := \frac{(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{2}}{8}$ $M_{S.4b.0} = 6.24\cdot kip\cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.5} := if\left(\frac{2}{3}Mcr_{4b.0} > M_{S.4b.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.5} = \text{"Cracks"}$

$M_{D.4b.0} := \frac{(wdl_{4b.mx})\cdot l_{4b}^{2}}{8}$ $M_{D.4b.0} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5\cdot Ig_{4b.0} = 141.6\cdot in^4$ $(0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0} = 208\cdot in^4$ $0.25\cdot Ig_{4b.0} = 70.8\cdot in^4$

$I_{cr.4b.0} := \min\left[0.5Ig_{4b.0}, \max\left[0.25\cdot Ig_{4b.0}, (0.1 + 25\cdot\rho_{4b.0a})\cdot\left(1.2 - 0.2\cdot\frac{b_{w.4a}}{d3_{4b.a0}}\right)\cdot Ig_{4b.0}\right]\right]$ $I_{cr.4b.0} = 141.6\cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4b.0} := if\left[M_{D.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{D.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4b.0} = 283.3\cdot in^4$

$I_{e.S.4b.0} := if\left[M_{S.4b.0} < \frac{2}{3}Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.0}}{M_{S.4b.0}}\right)^2\cdot\left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4b.0} = 145\cdot in^4$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4b.0} := \frac{5\cdot(wdl_{4b.mx})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4b.0} = 0.01471\cdot in$ $\Delta_{DL.ref} = \text{"DL Defl."}$

$\Delta_{S.4b.0} := \frac{5\cdot(wdl_{4b.mx} + wll_{4b.0})\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4b.0}}$ $\Delta_{S.4b.0} = 0.22564\cdot in$ $\Delta_{ref} = \text{"Service Load Defl."}$

$\Delta_{LL.4b.0} := \Delta_{S.4b.0} - \Delta_{DL.4b.0}$ $\Delta_{LL.4b.0} = 0.21093\cdot in$ $\Delta_{LL.ref} = \text{"LL Defl."}$

$\xi_3 = 1$ $\xi_{3.ref} = \text{"Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)"}$ Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref} = \text{"Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)"}$ Considered for Dead Load

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$\rho'_{4b.0a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.0.a} := \Delta_{DL.4b.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4b.0}\cdot\left(1 + \frac{\xi_3}{1 + 50\cdot\rho'_{4b.0a}}\right)$$

$\Delta_{LT.4b.0.a} = 0.32033\cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.0.a} := \frac{5\cdot\left[\left(wdl_{4b.mx} + wll_{4b.0}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \quad \Delta_{4b.0.a} = 0.115\cdot in \quad \Delta_{ref}$$ = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.0} = 0.467\cdot in \quad \delta_{4.ref}$$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\Delta_{4b.0} := \Delta_{4b.0.a} + \Delta_{LT.4b.0.a}$ $\Delta_{4b.0} = 0.436\cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$$\eta_{4b.0} := \frac{l_{4b}}{\Delta_{4b.0}} \quad \eta_{4b.0} = 257 \quad \eta_{ref}$$ = "Deflection Ratio"

$chk_{4b.6} := if\left(\Delta_{4b.0} < \delta_{4b.0}, "OK", "NG"\right)$ $chk_{4b.6}$ = "OK"

Strength, Floor Live Load Option 1, $w_{llf.1} = 200\cdot psf$

$$f_{4b.1a}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1a} := root\left(f_{4b.1a}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1a} = 2.22324\cdot in$

Depth of Compression Block

$$c_{4b.1a} := \frac{a_{4b.1a}}{\beta_1} \quad c_{4b.1a} = 2.695\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} = -34.016\cdot kip$ Concrete Compression Force

$$F1_{4b.1a} := A1_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d1_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F1_{4b.1a} = -1.984\cdot kip$$

$$F2_{4b.1a} := A2_{4b.a2}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d2_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F2_{4b.1a} = 0\cdot kip$$

$$F3_{4b.1a} := A3_{4b.a1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1a}\cdot\beta_1}\cdot d3_{4b.a1}\right)\cdot E_s, f_y\right], -f_y\right] \quad F3_{4b.1a} = 36\cdot kip$$

$-0.85\cdot f'_c\cdot b_{w.4a}\cdot a_{4b.1a} + F1_{4b.1a} + F2_{4b.1a} + F3_{4b.1a} = 0\cdot kip$

$$Mn_{4b.1a} := -0.85\cdot f'_c\cdot b_{w.4a}\cdot\frac{a_{4b.1a}^{2}}{2} + F1_{4b.1a}\cdot d1_{4b.a1} + F2_{4b.1a}\cdot d2_{4b.a1} + F3_{4b.1a}\cdot d3_{4b.a1} \quad Mn_{4b.1a} = 15.893\cdot kip\cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.1b}(ag_{4b}) := -0.85\cdot f'_c\cdot b_{w.4a}\cdot ag_{4b} + P_{u.4b.1.mn} \ldots + A1_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d1_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A2_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d2_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right] \ldots + A3_{4b.b1}\cdot\max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b}\cdot\beta_1}\cdot d3_{4b.b1}\right)\cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.1b} := root\left(f_{4b.1b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.1b} = 1.54766\cdot in$

Depth of Compression Block

$$c_{4b.1b} := \frac{a_{4b.1b}}{\beta_1} \quad c_{4b.1b} = 1.876\cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} = -23.679 \cdot kip$

$$F1_{4b.1b} := A1_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d1_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.1b} = 11.679 \cdot kip$$

$$F2_{4b.1b} := A2_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d2_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.1b} = 0 \cdot kip$$

$$F3_{4b.1b} := A3_{4b.b1} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.1b} \cdot \beta_1} \cdot d3_{4b.b1}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.1b} = 12 \cdot kip$$

$$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.1b} + F1_{4b.1b} + F2_{4b.1b} + F3_{4b.1b} = 0 \cdot kip$$

$$Mn_{4b.1b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.1b}^2}{2} + F1_{4b.1b} \cdot d1_{4b.b1} + F2_{4b.1b} \cdot d2_{4b.b1} + F3_{4b.1b} \cdot d3_{4b.b1} \qquad Mn_{4b.1b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.1a} := -0.003 + \frac{0.003}{c_{4b.1a}} \cdot d3_{4b.a1} \qquad \varepsilon_{4b.1a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.1b} := -0.003 + \frac{0.003}{c_{4b.1b}} \cdot d3_{4b.b1} \qquad \varepsilon_{4b.1b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.1a} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1a} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.1b} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{4b.1b} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.1b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.1a} := \phi_{4b.1a} \cdot A3_{4b.a1} \cdot f_y \cdot \left(d3_{4b.a1} - \frac{a_{4b.1a}}{2}\right) \qquad \phi Mn_{4b.1a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.1b} := \phi_{4b.1b} \cdot A3_{4b.b1} \cdot f_y \cdot \left(d3_{4b.b1} - \frac{a_{4b.1b}}{2}\right) \qquad \phi Mn_{4b.1b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$$\rho_{4b.1a} := \frac{A3_{4b.a1}}{d3_{4b.a1} \cdot b_{w.4a}} \qquad \rho_{4b.1a} = 0.0233 \qquad \rho_{4b.1b} := \frac{A3_{4b.b1}}{d3_{4b.b1} \cdot b_{w.4a}} \qquad \rho_{4b.1b} = 0.0078 \qquad \rho_{ref} = \text{"Reinf. Ratio"}$$

$$\rho'_{4b.1a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a1} = 0in, 0in^2, \text{if}\left(d1_{4b.a1} < a_{4b.1a}, A1_{4b.a1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.a1} = 0in, 0in^2, \text{if}\left(d2_{4b.a1} < a_{4b.1a}, A2_{4b.a2}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.a1} = 0in, 0in^2, \text{if}\left(d3_{4b.a1} < a_{4b.1a}, A3_{4b.a1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a1}} \qquad \rho'_{4b.1a} = 0.0308$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\rho'_{4b.1b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b1} = 0in, 0in^2, \text{if}\left(d1_{4b.b1} < a_{4b.1b}, A1_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d2_{4b.b1} = 0in, 0in^2, \text{if}\left(d2_{4b.b1} < a_{4b.1b}, A2_{4b.b1}, 0in^2\right)\right) \\ \text{if}\left(d3_{4b.b1} = 0in, 0in^2, \text{if}\left(d3_{4b.b1} < a_{4b.1b}, A3_{4b.b1}, 0in^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b1}} \qquad \rho'_{4b.1b} = 0$$

$$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$a1_{min} = 0.0034 \qquad a2_{min} = 0.0033 \qquad a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$$

$$A_{mn.4b.1} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a1}, d3_{4b.b1}\right) \qquad A_{mn.4b.1} = 0.086 \cdot in^2$$

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APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$chk_{4b.7} := if\left(A_{mn.4b.1} < \min\left(A1_{4b.a1}, A1_{4b.b1}\right), "OK", "NG"\right)$ $chk_{4b.7} = "OK"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear\ Strgth\ Reduc.\ Fact,\ Tbl\ (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod.\ Factor\ for\ Sand\ LW\ Conc.,\ Tbl\ (19.2.4.1(b))"$

$\lambda_{s.4b.1a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a1}, d3_{4b.b1}\right)}{10 \cdot in}}}\right)$ $\lambda_{s.4b.1a} = 1$ $\lambda_{s.ref} = "Size\ Effect\ Fact,\ Eqn\ (22.5.5.1.3)"$

$Vc_{4b.1.a} := 8 \cdot \lambda_{s.4b.1a} \cdot \lambda \cdot \rho_{4b.1a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$ $Vc_{4b.1.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)$

$Vc_{4b.1.a} = 3.32 \cdot kip$ $Vc_{4b.1.b} = 7.27 \cdot kip$ $\phi Vc_{4b.1} := \phi_V \cdot \min\left(Vc_{4b.1.a}, Vc_{4b.1.b}\right)$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$\phi Vc_{ref} = "Conc.\ Shear\ Strgth,\ Eqn\ (22.5.5.1)"$

$M_{u.4b.1a} = 7.923 \cdot kip \cdot ft$ $\phi Mn_{4b.1a} = 13.27 \cdot kip \cdot ft$ $M_{u.4b.1b} = 0.578 \cdot kip \cdot ft$ $\phi Mn_{4b.1b} = 5.041 \cdot kip \cdot ft$

$chk_{4b.8} := if\left(\phi Mn_{4b.1a} \geq M_{u.4b.1a}, "OK", "NG"\right)$ $chk_{4b.9} := if\left(\phi Mn_{4b.1b} \geq M_{u.4b.1b}, "OK", "NG"\right)$ $chk_{4b.8} = "OK"$ $chk_{4b.9} = "OK"$

$V_{u.4b.1} = 3.395 \cdot kip$ $2V_{u.4b.1} = 6.791 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$

$$chk_{4b.10} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.1} \cdot 0.5 \geq V_{u.4b.1} \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.1} \cdot 0.5 < V_{u.4b.1} \wedge \phi Vc_{4b.1} \geq V_{u.4b.1} \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$ $chk_{4b.10} = "Stirrups\ Required"$

$S_{4b.1} := floor\left(\frac{\min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{2} \cdot \frac{1}{in}\right) \cdot in$ $S_{4b.1} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$

$V_{s.4b.1} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a1}, d3_{4b.b1}\right)}{S_{4b.1}}$ $V_{s.4b.1} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4b.11} := if\left(V_{s.4b.1} \geq \frac{V_{u.4b.1}}{\phi_V} - \phi Vc_{4b.1}, "OK", "NG"\right)$ $chk_{4b.11} = "OK"$

$x_{stir.4b.1} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.1} - \max\left(Vu_{4b.1}\right)}{-\max\left(\left|w_{u.mx.4b.1}\right|, \left|w_{u.mn.4b.1}\right|\right)}\right)\right)$

$x_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$$l_{stir.4b.1} := \begin{cases} "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.1}\right) + S_{4b.1}\right] \cdot 2 \leq 0 \\ x_{stir.4b.1} & \text{otherwise} \end{cases}$$

$l_{stir.4b.1} = 14.9 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

$No_{stir.4b.1} := if\left(l_{stir.4b.1} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.1}}\right), ceil\left(\frac{l_{stir.4b.1}}{S_{4b.1}} + 1\right)\right)$

$No_{stir.4b.1} = 6$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = "Modulus\ of\ Rupture,\ Eqn\ (19.2.3.1)"$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = "Modulus\ of\ Elas.\ Conc."$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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L Beam Properties - $Beam_{4b.1.L} := \text{"L-Beam"}$

$b_{e1.L.4b.1} := b_{w.4a} + \frac{l_{4b}}{12}$ $b_{e1.L.4b.1} = 13.33 \cdot in$ $b_{e2.L.4b.1} := b_{w.4a} + 6 \cdot t_{slab}$ $b_{e2.L.4b.1} = 16 \cdot in$

$b_{e3.L.4b.1} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2}$ $b_{e3.L.4b.1} = 14 \cdot in$ $b_{ef.L.4b.1} := \min\left(b_{e1.L.4b.1}, b_{e2.L.4b.1}, b_{e3.L.4b.1}\right)$ $b_{ef.L.4b.1} = 13.33 \cdot in$

$$y_{t.L4.1} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}}$$

$y_{t.L4.1} = 5.033 \cdot in$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.1} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.1} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots + \frac{1}{12} \cdot b_{ef.L.4b.0} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.0} \cdot \left[y_{t.L4.1} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2$$

$Ig_{L.4b.1} = 283.3 \cdot in^4$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.1.R} := \text{"Rectangular"}$

$b_{ef.R.4b.1} := b_{w.4a}$ $b_{ef.R.4b.1} = 4 \cdot in$ $y_{t.R4.1} := \frac{t_{floor}}{2}$ $y_{t.R4.1} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Ig_{R.4b.1} := \frac{1}{12} \cdot b_{ef.R.4b.1} \cdot t_{floor}^3$ $Ig_{R.4b.1} = 170.7 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

Consider $Beam_{4b.1} := Beam_{4b.1.L}$ For Deflection

$b_{ef.4b.1} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.1} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.1} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$Mcr_{4b.1} := f_r \cdot \frac{Ig_{4b.1}}{y_{t.4b.1}}$ $Mcr_{4b.1} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref} = \text{"Crack Moment of Inertia, Eqn (24.2.3.5)"}$

$M_{S.4b.1} := \frac{\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^2}{8}$ $M_{S.4b.1} = 5.15 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4b.12} := \text{if}\left(\frac{2}{3} Mcr_{4b.1} > M_{S.4b.1}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.12} = \text{"Cracks"}$

$M_{D.4b.1} := \frac{\left(wdl_{4b.mx}\right) \cdot l_{4b}^2}{8}$ $M_{D.4b.1} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$0.5 \cdot Ig_{4b.1} = 141.6 \cdot in^4$ $\left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.1} = 70.8 \cdot in^4$

$$I_{cr.4b.1} := \min\left[0.5 Ig_{4b.1}, \max\left[0.25 \cdot Ig_{4b.1}, \left(0.1 + 25 \cdot \rho_{4b.1a}\right) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a1}}\right) \cdot Ig_{4b.1}\right]\right]$$

$I_{cr.4b.1} = 141.6 \cdot in^4$

$I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
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Plan No.: MET-4193-Standard Concrete
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Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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Email: info@modularconnections.co

$$I_{e.DL.4b.1} := \text{if}\left[M_{D.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.DL.4b.1} = 283.3 \cdot in^4$$

$$I_{e.S.4b.1} := \text{if}\left[M_{S.4b.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1 - \left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4b.1}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right] \qquad I_{e.S.4b.1} = 146.6 \cdot in^4$$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$$\Delta_{DL.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.1}} \qquad \Delta_{DL.4b.1} = 0.01471 \cdot in \qquad \Delta_{DL.ref} = \text{"DL Defl."}$$

$$\Delta_{S.4b.1} := \frac{5 \cdot \left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.1}} \qquad \Delta_{S.4b.1} = 0.18422 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\Delta_{LL.4b.1} := \Delta_{S.4b.1} - \Delta_{DL.4b.1} \qquad \Delta_{LL.4b.1} = 0.16951 \cdot in \qquad \Delta_{LL.ref} = \text{"LL Defl."}$$

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.1a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$$\Delta_{LT.4b.1.a} := \Delta_{DL.4b.1} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.1a}}\right) + \Delta_{LL.4b.1} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.1a}}\right) \qquad \Delta_{LT.4b.1.a} = 0.26259 \cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4b.1.a} := \frac{5 \cdot \left[\left(wdl_{4b.mx} + wll_{4b.1}\right) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.1}} \qquad \Delta_{4b.1.a} = 0.095 \cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4b.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4b.1} = 0.467 \cdot in \qquad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4b.1} := \Delta_{4b.1.a} + \Delta_{LT.4b.1.a} \qquad \Delta_{4b.1} = 0.358 \cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4b.1} := \frac{l_{4b}}{\Delta_{4b.1}} \qquad \eta_{4b.1} = 312.9 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$chk_{4b.13} := \text{if}\left(\Delta_{4b.1} < \delta_{4b.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.13}$ = "OK"

Strength, Floor Live Load Option 2, $w_{llf.2} = 150 \cdot psf$

$$f_{4b.2a}\left(ag_{4b}\right) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d1_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d2_{4b.a2} \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1}\right) \cdot d3_{4b.a2} \cdot E_s, f_y\right], -f_y\right]$$

$$a_{4b.2a} := \text{root}\left(f_{4b.2a}\left(ag_{4b}\right), ag_{4b}\right) \qquad a_{4b.2a} = 2.22324 \cdot in$$

Depth of Compression Block

$$c_{4b.2a} := \frac{a_{4b.2a}}{\beta_1} \qquad c_{4b.2a} = 2.695 \cdot in$$

c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"

Concrete Compression Force

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} = -34.016 \cdot kip$

$$F1_{4b.2a} := A1_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d1_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2a} = -1.984 \cdot kip$$

$$F2_{4b.2a} := A2_{4b.a3} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d2_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2a} = 0 \cdot kip$$

$$F3_{4b.2a} := A3_{4b.a2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2a} \cdot \beta_1} \cdot d3_{4b.a2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2a} = 36 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2a} + F1_{4b.2a} + F2_{4b.2a} + F3_{4b.2a} = 0 \cdot kip$

$$Mn_{4b.2a} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2a}^2}{2} + F1_{4b.2a} \cdot d1_{4b.a2} + F2_{4b.2a} \cdot d2_{4b.a2} + F3_{4b.2a} \cdot d3_{4b.a2} \qquad Mn_{4b.2a} = 15.893 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$f_{4b.2b}(ag_{4b}) := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot ag_{4b} + P_{u.4b.2.mn} \ldots$$
$$+ A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \ldots$$
$$+ A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{ag_{4b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right]$$

$a_{4b.2b} := \text{root}\left(f_{4b.2b}(ag_{4b}), ag_{4b}\right)$

$a_{4b.2b} = 1.54766 \cdot in$

Depth of Compression Block

$$c_{4b.2b} := \frac{a_{4b.2b}}{\beta_1} \qquad c_{4b.2b} = 1.876 \cdot in$$

$c_{ref} = \text{"Depth of N/A Axis, Eqn (22.2.2.4.1)"}$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} = -23.679 \cdot kip$

Concrete Compression Force

$$F1_{4b.2b} := A1_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d1_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F1_{4b.2b} = 11.679 \cdot kip$$

$$F2_{4b.2b} := A2_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d2_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F2_{4b.2b} = 0 \cdot kip$$

$$F3_{4b.2b} := A3_{4b.b2} \cdot \max\left[\min\left[\left(-0.003 + \frac{0.003}{a_{4b.2b} \cdot \beta_1} \cdot d3_{4b.b2}\right) \cdot E_s, f_y\right], -f_y\right] \qquad F3_{4b.2b} = 12 \cdot kip$$

$-0.85 \cdot f'_c \cdot b_{w.4a} \cdot a_{4b.2b} + F1_{4b.2b} + F2_{4b.2b} + F3_{4b.2b} = 0 \cdot kip$

$$Mn_{4b.2b} := -0.85 \cdot f'_c \cdot b_{w.4a} \cdot \frac{a_{4b.2b}^2}{2} + F1_{4b.2b} \cdot d1_{4b.b2} + F2_{4b.2b} \cdot d2_{4b.b2} + F3_{4b.2b} \cdot d3_{4b.b2} \qquad Mn_{4b.2b} = 6.369 \cdot kip \cdot ft$$

Nominal Flexural Strength, Max Moment

$$\varepsilon_{4b.2a} := -0.003 + \frac{0.003}{c_{4b.2a}} \cdot d3_{4b.a2} \qquad \varepsilon_{4b.2a} = 0.0042 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\varepsilon_{4b.2b} := -0.003 + \frac{0.003}{c_{4b.2b}} \cdot d3_{4b.b2} \qquad \varepsilon_{4b.2b} = 0.0072 \qquad \varepsilon_{ref} = \text{"Tensile Strain, Extreme Layer"}$$

$$\phi_{4b.2a} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2a} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2a} = 0.831 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi_{4b.2b} := \min\left[\max\left[0.65, 0.65 + (\varepsilon_{4b.2b} - 0.002) \cdot \left(\frac{250}{3}\right)\right], 0.9\right] \qquad \phi_{4b.2b} = 0.9 \qquad \phi_{ref} = \text{"Strgth Reduc. Fact, Tbl (21.2.1)"}$$

$$\phi Mn_{4b.2a} := \phi_{4b.2a} \cdot A3_{4b.a2} \cdot f_y \cdot \left(d3_{4b.a2} - \frac{a_{4b.2a}}{2}\right) \qquad \phi Mn_{4b.2a} = 13.27 \cdot k \cdot ft \qquad \phi M_{p.ref} = \text{"Flexural Strength, +ve, Sect (22.3)"}$$

$$\phi Mn_{4b.2b} := \phi_{4b.2b} \cdot A3_{4b.b2} \cdot f_y \cdot \left(d3_{4b.b2} - \frac{a_{4b.2b}}{2}\right) \qquad \phi Mn_{4b.2b} = 5.04 \cdot k \cdot ft \qquad \phi M_{n.ref} = \text{"Flexural Strength, -ve, Sect (22.3)"}$$

$\rho_{4b.2a} := \frac{A3_{4b.a2}}{d3_{4b.a2} \cdot b_{w.4a}}$ $\rho_{4b.2a} = 0.0233$ $\rho_{4b.2b} := \frac{A3_{4b.b2}}{d3_{4b.b2} \cdot b_{w.4a}}$ $\rho_{4b.2b} = 0.0078$ $\rho_{ref} = \text{"Reinf. Ratio"}$

$$\rho'_{4b.2a} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.a2} < a_{4b.2a}, A1_{4b.a2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.a2} < a_{4b.2a}, A2_{4b.a3}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.a2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.a2} < a_{4b.2a}, A3_{4b.a2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.a2}}$$

$\rho'_{4b.2a} = 0.0308$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$$\rho'_{4b.2b} := \frac{\sum \begin{pmatrix} \text{if}\left(d1_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d1_{4b.b2} < a_{4b.2b}, A1_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d2_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d2_{4b.b2} < a_{4b.2b}, A2_{4b.b2}, 0\text{in}^2\right)\right) \\ \text{if}\left(d3_{4b.b2} = 0\text{in}, 0\text{in}^2, \text{if}\left(d3_{4b.b2} < a_{4b.2b}, A3_{4b.b2}, 0\text{in}^2\right)\right) \end{pmatrix}}{b_{w.4a} \cdot d1_{4b.b2}}$$

$\rho'_{4b.2b} = 0$

$\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

$a1_{min} = 0.0034$ $a2_{min} = 0.0033$ $a1_{min.ref} = \text{"Min Area of Reinf. *bwd - Beam, Sec. (9.6.1.2a)"}$

$A_{mn.4b.2} := \max\left(a1_{min}, a2_{min}\right) \cdot b_{w.4a} \cdot \max\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $A_{mn.4b.2} = 0.086 \cdot \text{in}^2$

$chk_{4b.14} := \text{if}\left(A_{mn.4b.2} < \min\left(A1_{4b.a2}, A1_{4b.b2}\right), \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.14} = \text{"OK"}$

$\phi_V = 0.75$ $\phi_{V.ref} = \text{"Shear Strgth Reduc. Fact, Tbl (21.2.1)"}$

$\lambda = 0.85$ $\lambda_{ref} = \text{"Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"}$

$$\lambda_{s.4b.2a} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{\max\left(d3_{4b.a2}, d3_{4b.b2}\right)}{10 \cdot \text{in}}}}\right)$$ $\lambda_{s.4b.2a} = 1$ $\lambda_{s.ref} = \text{"Size Effect Fact, Eqn (22.5.5.1.3)"}$

$Vc_{4b.2.a} := 8 \cdot \lambda_{s.4b.2a} \cdot \lambda \cdot \rho_{4b.2a}^{\frac{1}{3}} \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$ $Vc_{4b.2.b} := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_{w.4a} \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)$

$Vc_{4b.2.a} = 3.32 \cdot \text{kip}$ $Vc_{4b.2.b} = 7.27 \cdot \text{kip}$ $\phi Vc_{4b.2} := \phi_V \cdot \min\left(Vc_{4b.2.a}, Vc_{4b.2.b}\right)$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$\phi Vc_{ref} = \text{"Conc. Shear Strgth, Eqn (22.5.5.1)"}$

$M_{u.4b.2a} = 6.181 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2a} = 13.27 \cdot \text{kip} \cdot \text{ft}$ $M_{u.4b.2b} = 0.578 \cdot \text{kip} \cdot \text{ft}$ $\phi Mn_{4b.2b} = 5.041 \cdot \text{kip} \cdot \text{ft}$

$chk_{4b.15} := \text{if}\left(\phi Mn_{4b.2a} \geq M_{u.4b.2a}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.15} = \text{"OK"}$

$chk_{4b.16} := \text{if}\left(\phi Mn_{4b.2b} \geq M_{u.4b.2b}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.16} = \text{"OK"}$

$V_{u.4b.2} = 2.649 \cdot \text{kip}$ $2V_{u.4b.2} = 5.298 \cdot \text{kip}$ $\phi Vc_{4b.2} = 2.492 \cdot \text{kip}$

$$chk_{4b.17} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq V_{u.4b.2} \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.2} \cdot 0.5 < V_{u.4b.2} \wedge \phi Vc_{4b.2} \geq V_{u.4b.2} \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases}$$

$chk_{4b.17} = \text{"Stirrups Required"}$

$$S_{4b.2} := \text{floor}\left(\frac{\min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{2} \cdot \frac{1}{\text{in}}\right) \cdot \text{in}$$ $S_{4b.2} = 3 \cdot \text{in}$ $S_{stir.ref} = \text{"Max Stirrup Spacing"}$

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$$V_{s.4b.2} := \frac{A_{4b.c.0} \cdot f_y \cdot \min\left(d3_{4b.a2}, d3_{4b.b2}\right)}{S_{4b.2}} \qquad V_{s.4b.2} = 14.03 \cdot kip \qquad Vs_{1way.ref} = \text{"1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"}$$

$$chk_{4b.18} := if\left(V_{s.4b.2} \geq \frac{V_{u.4b.2}}{\phi_V} - \phi Vc_{4b.2}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4b.18} = \text{"OK"}$$

$$x_{stir.4b.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(8.99in, \frac{\phi Vc_{4b.2} - \max\left(Vu_{4b.2}\right)}{-\max\left(\left|w_{u.mx.4b.2}\right|, \left|w_{u.mn.4b.2}\right|\right)}\right)\right)$$

$$x_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$l_{stir.4b.2} := \begin{cases} \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4b.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ x_{stir.4b.2} & \text{otherwise} \end{cases}$$

$$l_{stir.4b.2} = 9 \cdot in \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$$No_{stir.4b.2} := if\left(l_{stir.4b.2} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4b.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4b.2} = 4$ Min. Number of Stirrups

Servicability

$f_r = 427.6 \cdot psi$ $f_{r.ref} = \text{"Modulus of Rupture, Eqn (19.2.3.1)"}$ $E_c = 2838 \cdot ksi$ $E_{c.ref} = \text{"Modulus of Elas. Conc."}$

L Beam Properties - $Beam_{4b.2.L} := \text{"L-Beam"}$

$$b_{e1.L.4b.2} := b_{w.4a} + \frac{l_{4b}}{12} \qquad b_{e1.L.4b.2} = 13.33 \cdot in \qquad b_{e2.L.4b.2} := b_{w.4a} + 6 \cdot t_{slab} \qquad b_{e2.L.4b.2} = 16 \cdot in$$

$$b_{e3.L.4b.2} := b_{w.4a} + \frac{s_{t.rib} - b_{w.4a}}{2} \qquad b_{e3.L.4b.2} = 14 \cdot in \qquad b_{ef.L.4b.2} := \min\left(b_{e1.L.4b.2}, b_{e2.L.4b.2}, b_{e3.L.4b.2}\right) \qquad b_{ef.L.4b.2} = 13.33 \cdot in$$

$$y_{t.L4.2} := \frac{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \frac{\left(t_{floor} - t_{slab}\right)}{2} + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left(t_{floor} - \frac{t_{slab}}{2}\right)}{b_{4a} \cdot \left(t_{floor} - t_{slab}\right) + t_{slab} \cdot b_{w.4a}} \qquad y_{t.L4.2} = 5.033 \cdot in$$

$y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{L.4b.2} := \frac{1}{12} \cdot b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right)^3 + b_{w.4a} \cdot \left(t_{floor} - t_{slab}\right) \cdot \left[y_{t.L4.2} - \frac{\left(t_{floor} - t_{slab}\right)}{2}\right]^2 \ldots$$

$$+ \frac{1}{12} \cdot b_{ef.L.4b.2} \cdot t_{slab}^3 + t_{slab} \cdot b_{ef.L.4b.2} \cdot \left[y_{t.L4.2} - \left(t_{floor} - \frac{t_{slab}}{2}\right)\right]^2 \qquad Ig_{L.4b.2} = 283.3 \cdot in^4$$

$I_{g.ref} = \text{"Gross Moment of Inertia"}$

Rectangular Beam Properties - $Beam_{4b.2.R} := \text{"Rectangular"}$

$b_{ef.R.4b.2} := b_{w.4a}$ $b_{ef.R.4b.2} = 4 \cdot in$ $y_{t.R4.2} := \frac{t_{floor}}{2}$ $y_{t.R4.2} = 4 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

$$Ig_{R.4b.2} := \frac{1}{12} \cdot b_{ef.R.4b.2} \cdot t_{floor}^3 \qquad Ig_{R.4b.2} = 170.7 \cdot in^4 \qquad I_{g.ref} = \text{"Gross Moment of Inertia"}$$

Consider $Beam_{4b.2} := Beam_{4b.2.L}$ For Deflection

$b_{ef.4b.2} = 13.333 \cdot in$ Effective Flange Width $Ig_{4b.2} = 283.3 \cdot in^4$ $I_{g.ref} = \text{"Gross Moment of Inertia"}$

$y_{t.4b.2} = 5.033 \cdot in$ $y_{t.ref} = \text{"Dist., Centroid-Tension Face"}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
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$Mcr_{4b.2} := f_r \cdot \frac{Ig_{4b.2}}{y_{t.4b.2}}$ $Mcr_{4b.2} = 2.01 \cdot kip \cdot ft$ $M_{cr.ref}$ = "Crack Moment of Inertia, Eqn (24.2.3.5)"

$M_{S.4b.2} := \frac{(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^2}{8}$ $M_{S.4b.2} = 4.06 \cdot kip \cdot ft$ $M_{S.ref}$ = "Service Moment"

$chk_{4b.19} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4b.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4b.19}$ = "Cracks"

$M_{D.4b.2} := \frac{(wdl_{4b.mx}) \cdot l_{4b}^2}{8}$ $M_{D.4b.2} = 0.79 \cdot kip \cdot ft$ $M_{D.ref}$ = "Dead Load Moment"

$0.5 \cdot Ig_{4b.2} = 141.6 \cdot in^4$ $(0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2} = 208 \cdot in^4$ $0.25 \cdot Ig_{4b.2} = 70.8 \cdot in^4$

$I_{cr.4b.2} := \min\left[0.5 Ig_{4b.2}, \max\left[0.25 \cdot Ig_{4b.2}, (0.1 + 25 \cdot \rho_{4b.2a}) \cdot \left(1.2 - 0.2 \cdot \frac{b_{w.4a}}{d3_{4b.a2}}\right) \cdot Ig_{4b.2}\right]\right]$ $I_{cr.4b.2} = 141.6 \cdot in^4$

$I_{cr.bm.ref}$ = "Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"

$I_{e.DL.4b.2} := if\left[M_{D.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.DL.4b.2} = 283.3 \cdot in^4$

$I_{e.S.4b.2} := if\left[M_{S.4b.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4b.2}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right]$ $I_{e.S.4b.2} = 149.7 \cdot in^4$

$I_{e.ref}$ = "Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"

$\Delta_{DL.4b.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}}$ $\Delta_{DL.4b.2} = 0.01471 \cdot in$ $\Delta_{DL.ref}$ = "DL Defl."

$\Delta_{S.4b.2} := \frac{5 \cdot (wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4b.2}}$ $\Delta_{S.4b.2} = 0.14219 \cdot in$ Δ_{ref} = "Service Load Defl."

$\Delta_{LL.4b.2} := \Delta_{S.4b.2} - \Delta_{DL.4b.2}$ $\Delta_{LL.4b.2} = 0.12748 \cdot in$ $\Delta_{LL.ref}$ = "LL Defl."

$\xi_3 = 1$ $\xi_{3.ref}$ = "Time-Dep. Fact., 3 Mos., Tbl (24.2.4.1.3)" Considered for Live Load

$\xi_{60} = 2$ $\xi_{60.ref}$ = "Time-Dep. Fact., >= 60 Mos., Tbl (24.2.4.1.3)" Considered for Dead Load

$\rho'_{4b.2a} = 0.031$ ρ'_{ref} = "Comp. Reinf. Ratio"

$\Delta_{LT.4b.2.a} := \Delta_{DL.4b.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4b.2} \cdot \left(1 + \frac{\xi_3}{1 + 50 \cdot \rho'_{4b.2a}}\right)$ $\Delta_{LT.4b.2.a} = 0.204 \cdot in$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$\Delta_{4b.2.a} := \frac{5 \cdot \left[(wdl_{4b.mx} + wll_{4b.2}) \cdot l_{4b}^4\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}}$ $\Delta_{4b.2.a} = 0.075 \cdot in$ Δ_{ref} = "Service Load Defl."

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$\delta_{4b.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$ $\delta_{4b.2} = 0.467 \cdot in$ $\delta_{4.ref}$ = "Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"

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$\Delta_{4b.2} := \Delta_{4b.2.a} + \Delta_{LT.4b.2.a}$ $\Delta_{4b.2} = 0.279 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4b.2} := \frac{l_{4b}}{\Delta_{4b.2}}$ $\eta_{4b.2} = 401.2$ η_{ref} = "Deflection Ratio"

$chk_{4b.20} := if(\Delta_{4b.2} < \delta_{4b.2}, "OK", "NG")$ $chk_{4b.20}$ = "OK"

c. Floor Weight

$No_{tr.rib}^T = (4 \quad 5 \quad 6 \quad 7 \quad 8 \quad 9 \quad 10 \quad 10 \quad 11)$ $No_{tr.rib.ref}$ = "# of Width Floor Ribs"

$L_f^T = (11.29 \quad 13.29 \quad 15.29 \quad 17.29 \quad 19.29 \quad 21.29 \quad 23.29 \quad 25.29 \quad 27.29)$ ft $L_{f.ref}$ = "Length of Floor Panel"

$t_{floor} = 8 \cdot in$ $t_{floor.ref}$ = "Total Floor Thickness" $t_{slab} = 2 \cdot in$ $t_{slab.ref}$ = "Floor Slab Thickness"

$W_{f.b.sw} = 9.75 \cdot in$ $W_{f.b.sw.ref}$ = "Width of Floor SW Beam" $l_{4b} = 112 \cdot in$ Clear Span of Rib

$W_f = 10.96$ ft $W_{f.ref}$ = "Width of Floor Panel"

$b_{w.4a.a} = 5 \cdot in$ Avg. Long. Floor Rib Width

$W_{f.b.ew} = 9.75 \cdot in$ $W_{f.b.ew.ref}$ = "Width of Floor EW Beam" $No_{l.rib} = 2$ Number of Long. Floor Ribs

$wgt_{f.slab} := \gamma_c \cdot t_{slab}$ $wgt_{f.slab} = 19.67 \cdot psf$ $wgt_{f.slab.ref}$:= "Weight of Floor Slab"

Floor Transverse Ribs $wgt_{f.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (l_{4b}) \cdot b_{w.4a} \cdot No_{tr.rib}}{W_f \cdot L_f}$

Floor Longitudinal Ribs $wgt_{f.l.rib} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot (L_f - 2 \cdot W_{f.b.ew} - No_{tr.rib} \cdot b_{w.4a}) \cdot b_{w.4a.a} \cdot No_{l.rib}}{W_f \cdot L_f}$

Floor Beams $wgt_{f.ewb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot 2 \cdot W_{f.b.ew} \cdot W_f}{W_f \cdot L_f}$ $wgt_{f.swb} := \frac{\gamma_c \cdot (t_{floor} - t_{slab}) \cdot W_{f.b.sw} \cdot (L_f - 2 \cdot W_{f.b.ew})}{W_f \cdot L_f}$

$wgt_{flr} := wgt_{f.slab} + wgt_{f.rib} + wgt_{f.l.rib} + wgt_{f.ewb} + wgt_{f.swb}$ $wgt_{flr.ref}$:= "Min. Floor Uniform Weigths"

$DL_{fl} := wgt_{flr} + w_{misc.f}$ $DL_{flr.ref}$:= "Max. Floor Uniform Weigths"

$wgt_{f.rib}^T = (5.93 \quad 6.3 \quad 6.57 \quad 6.78 \quad 6.95 \quad 7.08 \quad 7.19 \quad 6.62 \quad 6.75) \cdot psf$ Floor Transverse Ribs

$wgt_{f.l.rib}^T = (3.31 \quad 3.38 \quad 3.42 \quad 3.46 \quad 3.49 \quad 3.51 \quad 3.53 \quad 3.61 \quad 3.62) \cdot psf$ Floor Longitudinal Ribs

$wgt_{f.ewb}^T = (8.49 \quad 7.21 \quad 6.27 \quad 5.54 \quad 4.97 \quad 4.5 \quad 4.12 \quad 3.79 \quad 3.51) \cdot psf$ Floor EW Beams

$wgt_{f.swb}^T = (3.74 \quad 3.84 \quad 3.91 \quad 3.96 \quad 4.01 \quad 4.04 \quad 4.07 \quad 4.09 \quad 4.11) \cdot psf$ Floor SW Beams

$wgt_{flr}^T = (41.1 \quad 40.4 \quad 39.8 \quad 39.4 \quad 39.1 \quad 38.8 \quad 38.6 \quad 37.8 \quad 37.7) \cdot psf$ $wgt_{flr.ref}$ = "Min. Floor Uniform Weigths"

$DL_{fl}^T = (45.1 \quad 44.4 \quad 43.8 \quad 43.4 \quad 43.1 \quad 42.8 \quad 42.6 \quad 41.8 \quad 41.7) \cdot psf$ $DL_{flr.ref}$ = "Max. Floor Uniform Weigths"

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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d. Concentrated Loads

$P_{4d} := 1450lbf$, Over 6.25sf, per ASCE 2.5' x 2.5'. If concentrated load footprint is 2.5' x 2.5' , $b_{4d} := 2.5ft$

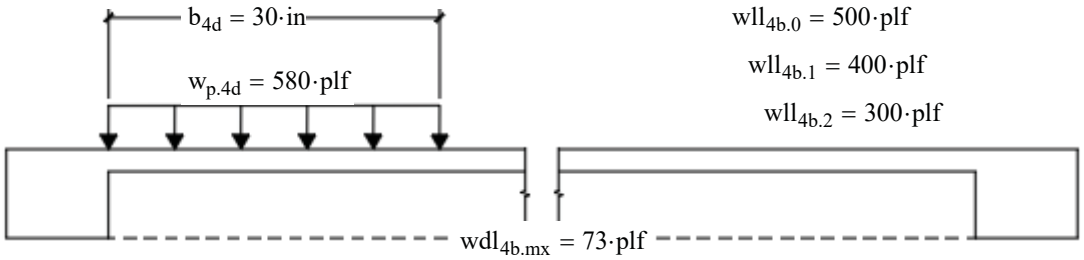
$chk_{4d.1} := if(b_{4d} \geq s_{t.rib}, "OK", "NG")$ $chk_{4d.1} = "OK"$

$if(chk_{4d.1} = "OK", "Load Is On A Rib", "Load Is ALL on Slab") = "Load Is On A Rib"$

Conservatively considering all the concentrated load is acting on 1 rib.

$\phi Vc_{4b.0} = 2.492 \cdot kip$ $\phi Vc_{4b.1} = 2.492 \cdot kip$ $\phi Vc_{4b.2} = 2.492 \cdot kip$

$w_{p.4d} := \frac{P_{4d}}{b_{4d}}$ $w_{p.4d} = 580 \cdot plf$



$$Vu_{x.4d.0}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.0} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.0}(0) = 5.875 \cdot kip$

$2\,Vu_{x.4d.0}(0) = 11.751 \cdot kip$

$$chk_{4d.2} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.0} \cdot 0.5 \geq Vu_{x.4d.0}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.0} \cdot 0.5 < Vu_{x.4d.0}(0) \wedge \phi Vc_{4b.0} \geq Vu_{x.4d.0}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.2} = "Stirrups\ Required"$

$S_{4b.0} = 3 \cdot in$ $S_{stir.ref} = "Max\ Stirrup\ Spacing"$ $V_{s.4b.0} = 14.03 \cdot kip$ $Vs_{1way.ref} = "1\text{-}Way\ Shear\ Reinf.\ Design,\ Eqn\ (22.5.8.5.3)"$

$chk_{4d.3} := if\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.0}(0)}{\phi_V} - \phi Vc_{4b.0}, "OK", "NG"\right)$ $chk_{4d.3} = "OK"$

$x_{stir.4d.0} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.0}, \frac{\phi Vc_{4b.0} - Vu_{x.4d.0}(0)}{-\frac{2\,Vu_{x.4d.0}(0)}{l_{4b}}}\right)\right)$

$x_{stir.4d.0} = 32.3 \cdot in$ $l_{stir.ref} = "Stirrup\ Min\ Cutoff\ Dist."$

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$l_{stir.4d.0} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.0} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.0}\right) + S_{4b.0}\right] \cdot 2 \le 0 \\ \max\left(x_{stir.4b.0}, x_{stir.4d.0}\right) & \text{otherwise} \end{cases}$ $l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$

$l_{stir.4d.0} = 32.3 \cdot in$

$No_{stir.4d.0} := if\left(l_{stir.4d.0} = \text{"Continuous"}, floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.0}}{S_{4b.0}}\right), ceil\left(\frac{l_{stir.4d.0}}{S_{4b.0}} + 1\right)\right)$ $No_{stir.4d.0} = 12$ Min. Number of Stirrups

$M_{u.4d.0} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{u.4d.0} = 10.312 \cdot kip \cdot ft$

$chk_{4d.4} := if\left(\phi Mn_{4b.0a} \ge M_{u.4d.0}, \text{"OK"}, \text{"NG"}\right)$ $chk_{4b.2} = \text{"OK"}$

$M_{S.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} + \frac{wll_{4b.0} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$ $M_{S.4d.0} = 6.64 \cdot kip \cdot ft$ $M_{S.ref} = \text{"Service Moment"}$

$chk_{4d.5} := if\left(\frac{2}{3} Mcr_{4b.0} > M_{S.4d.0}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right)$ $chk_{4d.5} = \text{"Cracks"}$

$M_{D.4d.0} := \frac{wdl_{4b.mx} \cdot l_{4b}^2}{8}$ $M_{D.4d.0} = 0.79 \cdot kip \cdot ft$ $M_{D.ref} = \text{"Dead Load Moment"}$

$I_{cr.4b.0} = 141.6 \cdot in^4$ $I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$

$I_{e.DL.4d.0} := if\left[M_{D.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{D.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.DL.4d.0} = 283.3 \cdot in^4$

$I_{e.S.4d.0} := if\left[M_{S.4d.0} < \frac{2}{3} Mcr_{4b.0}, Ig_{4b.0}, \frac{I_{cr.4b.0}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.0}}{M_{S.4d.0}}\right)^2 \cdot \left(1 - \frac{I_{cr.4b.0}}{Ig_{4b.0}}\right)}\right]$ $I_{e.S.4d.0} = 144.6 \cdot in^4$ $I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$\Delta_{DL.4d.0} := \frac{5 \cdot \left(wdl_{4b.mx}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.0}}$ $\Delta_{DL.4d.0} = 0.01471 \cdot in$ $\Delta_{S.4d.0} := \frac{5 \cdot \left(\frac{8 M_{S.4d.0}}{l_{4b}^2}\right) \cdot l_{4b}^4}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.0}}$ $\Delta_{S.4d.0} = 0.24093 \cdot in$

$\Delta_{LL.4d.0} := \Delta_{S.4d.0} - \Delta_{DL.4d.0}$ $\Delta_{LL.4d.0} = 0.226 \cdot in$ $\rho'_{4b.0a} = 0.031$ $\rho'_{ref} = \text{"Comp. Reinf. Ratio"}$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$\Delta_{LT.4d.0} := \Delta_{DL.4d.0}\cdot\left(1 + \frac{\xi_{60}}{1 + 50\cdot\rho'_{4b.0a}}\right) + \Delta_{LL.4d.0}\cdot\left(1 + \frac{\xi_{3}}{1 + 50\cdot\rho'_{4b.0a}}\right) \qquad \Delta_{LT.4d.0} = 0.34163\cdot in$$

$\Delta_{LT.ref}$ = "Time Dep. Defl."

$$\Delta_{4d.0.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.0}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.0}} \qquad \Delta_{4d.0.a} = 0.123\cdot in \qquad \Delta_{ref} = \text{"Service Load Defl."}$$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref}$ = "Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"

$$\delta_{4d.0} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \qquad \delta_{4d.0} = 0.467\cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.0} := \Delta_{4d.0.a} + \Delta_{LT.4d.0} \qquad \Delta_{4d.0} = 0.465\cdot in \qquad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.0} := \frac{l_{4b}}{\Delta_{4d.0}} \qquad \eta_{4d.0} = 241.1 \qquad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.6} := \text{if}\left(\Delta_{4b.0} < \delta_{4d.0}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.6} = \text{"OK"}$$

$$Vu_{x.4d.1}(x) := LF_{dead}\cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live}\cdot\left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right] \ldots + LF_{live}\cdot\left[wll_{4b.1}\cdot\left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d}\cdot b_{4d}}{2\cdot l_{4b}}\cdot\left(2\cdot l_{4b} - b_{4d}\right) - w_{p.4d}\cdot x\right]\right]$$

$Vu_{x.4d.1}(0) = 4.782\cdot kip$

$2\,Vu_{x.4d.1}(0) = 9.564\cdot kip$

$$chk_{4d.7} := \begin{cases} \text{"OK"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 \geq Vu_{x.4d.1}(0) \\ \text{"Min. Stirrups Required"} & \text{if } \phi Vc_{4b.1}\cdot 0.5 < Vu_{x.4d.1}(0) \wedge \phi Vc_{4b.1} \geq Vu_{x.4d.1}(0) \\ \text{"Stirrups Required"} & \text{otherwise} \end{cases} \qquad chk_{4d.7} = \text{"Stirrups Required"}$$

$S_{4b.1} = 3\cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.1} = 14.03\cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$$chk_{4d.8} := \text{if}\left(V_{s.4b.0} \geq \frac{Vu_{x.4d.1}(0)}{\phi_V} - \phi Vc_{4b.1}, \text{"OK"}, \text{"NG"}\right) \qquad chk_{4d.8} = \text{"OK"}$$

$$x_{stir.4d.1} := \text{if}\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2\cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.1}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.1}(0)}{-\frac{2\,Vu_{x.4d.1}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.1} = 26.8\cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.1} := \begin{cases} \text{"Continuous"} & \text{if } l_{stir.4b.1} = \text{"Continuous"} \\ \text{"Continuous"} & \text{if } W_f - 2\cdot W_{f.b.sw} - \left[\left(x_{stir.4d.1}\right) + S_{4b.1}\right]\cdot 2 \leq 0 \\ \max\left(x_{stir.4b.1}, x_{stir.4d.1}\right) & \text{otherwise} \end{cases} \qquad l_{stir.ref} = \text{"Stirrup Min Cutoff Dist."}$$

$l_{stir.4d.1} = 26.8\cdot in$

$$No_{stir.4d.1} := \text{if}\left(l_{stir.4d.1} = \text{"Continuous"}, \text{floor}\left(\frac{W_f - 2\cdot W_{f.b.sw} - S_{4b.1}}{S_{4b.0}}\right), \text{ceil}\left(\frac{l_{stir.4d.1}}{S_{4b.1}} + 1\right)\right)$$

$No_{stir.4d.1} = 10$ Min. Number of Stirrups

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$$M_{u.4d.1} := LF_{dead}\cdot\frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} \ldots + LF_{live}\cdot\frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} \ldots + LF_{live}\cdot\frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{u.4d.1} = 9.378\cdot kip\cdot ft$

$chk_{4d.9} := if\left(\phi Mn_{4b.1a} \geq M_{u.4d.1}, "OK", "NG"\right)$ $chk_{4d.9} = "OK"$

$$M_{S.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8} + \frac{b_{4d}\cdot w_{p.4d}\cdot\left(2\cdot l_{4b}-b_{4d}\right)}{8} + \frac{wll_{4b.1}\cdot\left(b_{4d}-l_{4b}\right)^{2}}{8}$$

$M_{S.4d.1} = 6.06\cdot kip\cdot ft$ $M_{S.ref} = "Service\ Moment"$

$$chk_{4d.10} := if\left(\frac{2}{3}Mcr_{4b.1} > M_{S.4d.1}, "Cracks\ Unlikely", "Cracks"\right)$$

$chk_{4d.10} = "Cracks"$

$$M_{D.4d.1} := \frac{wdl_{4b.mx}\cdot l_{4b}^{2}}{8}$$

$M_{D.4d.1} = 0.79\cdot kip\cdot ft$ $M_{D.ref} = "Dead\ Load\ Moment"$

$I_{cr.4b.1} = 141.6\cdot in^{4}$ $I_{cr.bm.ref} = "Crack\ Mom\ of\ Inertia\ -\ Beam,\ Tbl\ (6.6.3.1.1a)"$

$$I_{e.DL.4d.1} := if\left[M_{D.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{D.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.DL.4d.1} = 283.3\cdot in^{4}$

$$I_{e.S.4d.1} := if\left[M_{S.4d.1} < \frac{2}{3}Mcr_{4b.1}, Ig_{4b.1}, \frac{I_{cr.4b.1}}{1-\left(\frac{\frac{2}{3}Mcr_{4b.1}}{M_{S.4d.1}}\right)^{2}\cdot\left(1-\frac{I_{cr.4b.1}}{Ig_{4b.1}}\right)}\right]$$

$I_{e.S.4d.1} = 145.2\cdot in^{4}$ $I_{e.ref} = "Effec.\ Mom\ of\ Inertia,\ Tbl\ (6.6.3.1.1b)"$

$$\Delta_{DL.4d.1} := \frac{5\cdot\left(wdl_{4b.mx}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.DL.4b.1}}$$

$\Delta_{DL.4b.1} = 0.01471\cdot in$

$$\Delta_{S.4d.1} := \frac{5\cdot\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}}{384\cdot E_{c.spec}\cdot I_{e.S.4d.1}}$$

$\Delta_{S.4d.1} = 0.21884\cdot in$

$\Delta_{LL.4d.1} := \Delta_{S.4d.1} - \Delta_{DL.4d.1}$ $\Delta_{LL.4d.1} = 0.204\cdot in$ $\rho'_{4b.1a} = 0.031$ $\rho'_{ref} = "Comp.\ Reinf.\ Ratio"$

$$\Delta_{LT.4d.1} := \Delta_{DL.4d.1}\cdot\left(1+\frac{\xi_{60}}{1+50\cdot\rho'_{4b.1a}}\right) + \Delta_{LL.4d.1}\cdot\left(1+\frac{\xi_{3}}{1+50\cdot\rho'_{4b.1a}}\right)$$

$\Delta_{LT.4d.1} = 0.31085\cdot in$

$\Delta_{LT.ref} = "Time\ Dep.\ Defl."$

$$\Delta_{4d.1.a} := \frac{5\cdot\left[\left(\frac{8M_{S.4d.1}}{l_{4b}^{2}}\right)\cdot l_{4b}^{4}\right]}{384\cdot E_{c.spec}\cdot Ig_{4b.1}}$$

$\Delta_{4b.1.a} = 0.095\cdot in$ $\Delta_{ref} = "Service\ Load\ Defl."$

$\eta_{a.RF.NonDam} = 240$ $\eta_{a.RF.NonDam.ref} = "Allow\ Defl\ Ratio,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

$$\delta_{4d.1} := \frac{l_{4b}}{\eta_{a.RF.NonDam}}$$

$\delta_{4b.1} = 0.467\cdot in$ $\delta_{4.ref} = "Allow\ Defl.,\ Roof/Floor,\ Non-Dam.,\ Tbl\ (24.2.2)"$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$\Delta_{4d.1} := \Delta_{4d.1.a} + \Delta_{LT.4d.1}$ $\Delta_{4d.1} = 0.423 \cdot in$ $\Delta_{LTot.ref}$ = "Defl., Time Dep. + Serv"

$\eta_{4d.1} := \dfrac{l_{4b}}{\Delta_{4d.1}}$ $\eta_{4d.1} = 264.8$ η_{ref} = "Deflection Ratio"

$chk_{4d.11} := if(\Delta_{4b.1} < \delta_{4b.1}, "OK", "NG")$ $chk_{4d.11}$ = "OK"

$$Vu_{x.4d.2}(x) := LF_{dead} \cdot wdl_{4b.mx}\left(\frac{l_{4b}}{2} - x\right) \ldots + LF_{live} \cdot \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right] \ldots + LF_{live} \cdot \left[wll_{4b.2} \cdot \left(l_{4b} - b_{4d}\right) - \left[\frac{w_{p.4d} \cdot b_{4d}}{2 \cdot l_{4b}} \cdot \left(2 \cdot l_{4b} - b_{4d}\right) - w_{p.4d} \cdot x\right]\right]$$

$Vu_{x.4d.2}(0) = 3.689 \cdot kip$

$2\,Vu_{x.4d.2}(0) = 7.378 \cdot kip$

$$chk_{4d.12} := \begin{cases} "OK" & \text{if } \phi Vc_{4b.2} \cdot 0.5 \geq Vu_{x.4d.2}(0) \\ "Min.\ Stirrups\ Required" & \text{if } \phi Vc_{4b.2} \cdot 0.5 < Vu_{x.4d.2}(0) \wedge \phi Vc_{4b.2} \geq Vu_{x.4d.2}(0) \\ "Stirrups\ Required" & \text{otherwise} \end{cases}$$

$chk_{4d.12}$ = "Stirrups Required"

$S_{4b.2} = 3 \cdot in$ $S_{stir.ref}$ = "Max Stirrup Spacing" $V_{s.4b.2} = 14.03 \cdot kip$ $Vs_{1way.ref}$ = "1-Way Shear Reinf. Design, Eqn (22.5.8.5.3)"

$chk_{4d.13} := if\left(V_{s.4b.0} \geq \dfrac{Vu_{x.4d.2}(0)}{\phi_V} - \phi Vc_{4b.2}, "OK", "NG"\right)$ $chk_{4d.13}$ = "OK"

$$x_{stir.4d.2} := if\left(l_{4b} = l_{4b\_B}, \frac{W_f - 2 \cdot W_{f.b.sw}}{2} - 1.5in, \max\left(x_{stir.4b.2}, \frac{\phi Vc_{4b.1} - Vu_{x.4d.2}(0)}{-\dfrac{2\,Vu_{x.4d.2}(0)}{l_{4b}}}\right)\right)$$

$x_{stir.4d.2} = 18.2 \cdot in$ $l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$$l_{stir.4d.2} := \begin{cases} "Continuous" & \text{if } l_{stir.4b.2} = "Continuous" \\ "Continuous" & \text{if } W_f - 2 \cdot W_{f.b.sw} - \left[\left(x_{stir.4d.2}\right) + S_{4b.2}\right] \cdot 2 \leq 0 \\ \max\left(x_{stir.4b.2}, x_{stir.4d.2}\right) & \text{otherwise} \end{cases}$$

$l_{stir.ref}$ = "Stirrup Min Cutoff Dist."

$l_{stir.4d.2} = 18.2 \cdot in$

$$No_{stir.4d.2} := if\left(l_{stir.4d.2} = "Continuous", floor\left(\frac{W_f - 2 \cdot W_{f.b.sw} - S_{4b.2}}{S_{4b.2}}\right), ceil\left(\frac{l_{stir.4d.2}}{S_{4b.2}} + 1\right)\right)$$

$No_{stir.4d.2} = 8$ Min. Number of Stirrups

$$M_{u.4d.2} := LF_{dead} \cdot \frac{wdl_{4b.mx} \cdot l_{4b}^{\,2}}{8} \ldots + LF_{live} \cdot \frac{b_{4d} \cdot w_{p.4d} \cdot \left(2 \cdot l_{4b} - b_{4d}\right)}{8} \ldots + LF_{live} \cdot \frac{wll_{4b.2} \cdot \left(b_{4d} - l_{4b}\right)^2}{8}$$

$M_{u.4d.2} = 8.444 \cdot kip \cdot ft$

$chk_{4d.14} := if\left(\phi Mn_{4b.2a} \geq M_{u.4d.2}, "OK", "NG"\right)$ $chk_{4d.14}$ = "OK"

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$M_{S.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} + \frac{b_{4d} \cdot w_{p.4d} \cdot (2 \cdot l_{4b} - b_{4d})}{8} + \frac{wll_{4b.2} \cdot (b_{4d} - l_{4b})^{2}}{8} \quad M_{S.4d.2} = 5.48 \cdot kip \cdot ft \quad M_{S.ref} = \text{"Service Moment"}$$

$$chk_{4d.15} := if\left(\frac{2}{3} Mcr_{4b.2} > M_{S.4d.2}, \text{"Cracks Unlikely"}, \text{"Cracks"}\right) \quad chk_{4d.15} = \text{"Cracks"}$$

$$M_{D.4d.2} := \frac{wdl_{4b.mx} \cdot l_{4b}^{2}}{8} \quad M_{D.4d.2} = 0.79 \cdot kip \cdot ft \quad M_{D.ref} = \text{"Dead Load Moment"}$$

$$I_{cr.4b.2} = 141.6 \cdot in^{4} \quad I_{cr.bm.ref} = \text{"Crack Mom of Inertia - Beam, Tbl (6.6.3.1.1a)"}$$

$$I_{e.DL.4d.2} := if\left[M_{D.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{D.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.DL.4d.2} = 283.3 \cdot in^{4}$$

$$I_{e.S.4d.2} := if\left[M_{S.4d.2} < \frac{2}{3} Mcr_{4b.2}, Ig_{4b.2}, \frac{I_{cr.4b.2}}{1 - \left(\frac{\frac{2}{3} Mcr_{4b.2}}{M_{S.4d.2}}\right)^{2} \cdot \left(1 - \frac{I_{cr.4b.2}}{Ig_{4b.2}}\right)}\right] \quad I_{e.S.4d.2} = 146 \cdot in^{4}$$

$I_{e.ref} = \text{"Effec. Mom of Inertia, Tbl (6.6.3.1.1b)"}$

$$\Delta_{DL.4d.2} := \frac{5 \cdot (wdl_{4b.mx}) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.DL.4b.2}} \quad \Delta_{DL.4b.2} = 0.01471 \cdot in \quad \Delta_{S.4d.2} := \frac{5 \cdot \left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}}{384 \cdot E_{c.spec} \cdot I_{e.S.4d.2}} \quad \Delta_{S.4d.2} = 0.19666 \cdot in$$

$$\Delta_{LL.4d.2} := \Delta_{S.4d.2} - \Delta_{DL.4d.2} \quad \Delta_{LL.4d.2} = 0.182 \cdot in \quad \rho'_{4b.2a} = 0.031 \quad \rho'_{ref} = \text{"Comp. Reinf. Ratio"}$$

$$\Delta_{LT.4d.2} := \Delta_{DL.4d.2} \cdot \left(1 + \frac{\xi_{60}}{1 + 50 \cdot \rho'_{4b.2a}}\right) + \Delta_{LL.4d.2} \cdot \left(1 + \frac{\xi_{3}}{1 + 50 \cdot \rho'_{4b.2a}}\right) \quad \Delta_{LT.4d.2} = 0.27992 \cdot in \quad \Delta_{LT.ref} = \text{"Time Dep. Defl."}$$

$$\Delta_{4d.2.a} := \frac{5 \cdot \left[\left(\frac{8 M_{S.4d.2}}{l_{4b}^{2}}\right) \cdot l_{4b}^{4}\right]}{384 \cdot E_{c.spec} \cdot Ig_{4b.2}} \quad \Delta_{4d.2.a} = 0.101 \cdot in \quad \Delta_{ref} = \text{"Service Load Defl."}$$

$$\eta_{a.RF.NonDam} = 240 \quad \eta_{a.RF.NonDam.ref} = \text{"Allow Defl Ratio, Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\delta_{4d.2} := \frac{l_{4b}}{\eta_{a.RF.NonDam}} \quad \delta_{4b.2} = 0.467 \cdot in \quad \delta_{4.ref} = \text{"Allow Defl., Roof/Floor, Non-Dam., Tbl (24.2.2)"}$$

$$\Delta_{4d.2} := \Delta_{4d.2.a} + \Delta_{LT.4d.2} \quad \Delta_{4d.2} = 0.381 \cdot in \quad \Delta_{LTot.ref} = \text{"Defl., Time Dep. + Serv"}$$

$$\eta_{4d.2} := \frac{l_{4b}}{\Delta_{4d.2}} \quad \eta_{4d.2} = 293.8 \quad \eta_{ref} = \text{"Deflection Ratio"}$$

$$chk_{4d.16} := if(\Delta_{4d.2} < \delta_{4b.2}, \text{"OK"}, \text{"NG"}) \quad chk_{4d.16} = \text{"OK"}$$

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Const. Type: VB
Occupancy: S2
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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
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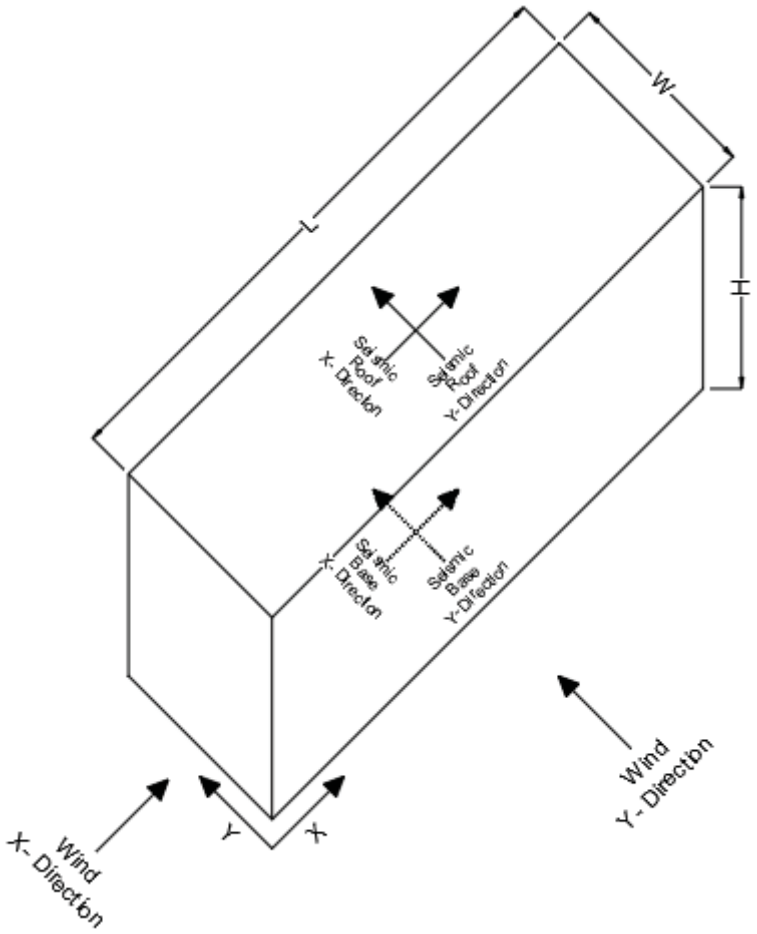
5) Lateral



a. Loads -

Weights

$Wt_{rf.mx} := DL_{r.mx} \cdot (L_r) \cdot (W_r)$

$Wt_{rf.mn} := DL_{r.mn} \cdot (L_r) \cdot (W_r)$

$w_{llr.eq} := if(w_{llr} \geq 30psf, 0.25 \cdot w_{llr}, 0psf)$

$w_{llr.eq} = 8.8 \cdot psf$

$Wt_{25ll} := w_{llr.eq} \cdot [(L_r) \cdot (W_r)]$

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Fire Rating of Ext. Walls:	2 HR
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$Wt_{rf.mx}^{T} = (7.9 \quad 9.2 \quad 10.5 \quad 11.8 \quad 13.1 \quad 14.4 \quad 15.7 \quad 17 \quad 18.3) \cdot kip$

Max Roof Weight, $w_{llr} = 35 \cdot psf$

$Wt_{25ll}^{T} = (1.3 \quad 1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.6 \quad 2.8 \quad 3) \cdot kip$

25% Roof LLr 0, $w_{llr} = 35 \cdot psf$

$Wt_{EQ.r} := Wt_{rf.mx} + Wt_{25ll}$ $Wt_{EQ.r}^{T} = (9 \quad 11 \quad 12 \quad 14 \quad 15 \quad 17 \quad 18 \quad 20 \quad 21) \cdot kip$

Seismic Weight of Roof

$Wt_{rf.mn}^{T} = (6.9 \quad 8 \quad 9.2 \quad 10.3 \quad 11.4 \quad 12.5 \quad 13.6 \quad 14.8 \quad 15.9) \cdot kip$

Min Roof Weight, $w_{llr} = 35 \cdot psf$

$h_{wl\_fl} := 0.5in$ Dist., Bottom of floor Panel to Bottom of Wall Panel $t_{key} = 0.75 \cdot in$ $t_{key.ref}$ = "Keyway thickness"

$el_{bot.rf} := H_{wall} + h_{wl\_fl} - \frac{2}{3} \cdot t_{key}$ $el_{bot.rf}^{T} = (108 \quad 120 \quad 128) \cdot in$ $el_{top.rf} := el_{bot.rf} + t_{r.r}$ $el_{top.rf}^{T} = (113.5 \quad 125.5 \quad 133.5) \cdot in$

$$el_{avg.rf} := \begin{pmatrix} mean(el_{bot.rf_0}, el_{top.rf_0}) \\ mean(el_{bot.rf_1}, el_{top.rf_1}) \\ mean(el_{bot.rf_2}, el_{top.rf_2}) \end{pmatrix} \quad el_{avg.rf} = \begin{pmatrix} 110.75 \\ 122.75 \\ 130.75 \end{pmatrix} \cdot in$$

Average Roof Elevation

$W\_z_{rh0} := Wt_{EQ.r} \cdot el_{avg.rf_0}$ $W\_z_{rh1} := Wt_{EQ.r} \cdot el_{avg.rf_1}$ $W\_z_{rh2} := Wt_{EQ.r} \cdot el_{avg.rf_2}$

Max Weight of Walls

$Wt_{w.h0.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mx} := (wgt_3 + w_{misc.w}) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

SeismicWeight of Walls

$Wt_{EQ.w.h0} := Wt_{w.h0.mx}$

$Wt_{EQ.w.h1} := Wt_{w.h1.mx}$

$Wt_{EQ.w.h2} := Wt_{w.h2.mx}$

Min Weight of Walls

$Wt_{w.h0.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h1.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h2.mn} := (wgt_3) \cdot H_{wall_0} \cdot 2 \cdot (W_{ew} + L_f)$

$Wt_{w.h0.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mx}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Max Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{w.h0.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{w.h1.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{w.h2.mn}^T = (16.3 \quad 17.7 \quad 19.1 \quad 20.5 \quad 21.9 \quad 23.3 \quad 24.8 \quad 26.2 \quad 27.6) \cdot kip$ Min Wall Weight, $H_{wall_2} = 10.7\,ft$

$Wt_{EQ.w.h0}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_0} = 9\,ft$

$Wt_{EQ.w.h1}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_1} = 10\,ft$

$Wt_{EQ.w.h2}^T = (18.7 \quad 20.4 \quad 22 \quad 23.6 \quad 25.3 \quad 26.9 \quad 28.5 \quad 30.2 \quad 31.8) \cdot kip$ Seismic Wall Weight, $H_{wall_2} = 10.7\,ft$

$el_{top.w} := h_{wl\_fl} + H_{wall}$ $\quad el_{top.w}^T = (108.5 \quad 120.5 \quad 128.5) \cdot in$ Height of top of Wall Above Bottom of Floor

$$el_{avg.w} := \begin{pmatrix} mean\left(h_{wl\_fl}, el_{top.w_0}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_1}\right) \\ mean\left(h_{wl\_fl}, el_{top.w_2}\right) \end{pmatrix} \qquad el_{avg.w} = \begin{pmatrix} 54.5 \\ 60.5 \\ 64.5 \end{pmatrix} \cdot in$$

Height of Center of Mass of Wall Above Bottom of Floor

$W\_z_{w.h0} := Wt_{EQ.w.h0} \cdot el_{avg.w_0}$ $\quad W\_z_{w.h1} := Wt_{EQ.w.h1} \cdot el_{avg.w_1}$ $\quad W\_z_{w.h2} := Wt_{EQ.w.h2} \cdot el_{avg.w_2}$

$w_{llf.0.eq} := if\left(w_{llf.0} \geq 100psf, 0.25 \cdot w_{llf.0}, 0psf\right)$ $\quad w_{llf.0.eq} = 62.5 \cdot psf$ $\quad Wt_{25llf0} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.1.eq} := if\left(w_{llf.1} \geq 100psf, 0.25 \cdot w_{llf.1}, 0psf\right)$ $\quad w_{llf.1.eq} = 50 \cdot psf$ $\quad Wt_{25llf1} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$w_{llf.2.eq} := if\left(w_{llf.2} \geq 100psf, 0.25 \cdot w_{llf.2}, 0psf\right)$ $\quad w_{llf.2.eq} = 37.5 \cdot psf$ $\quad Wt_{25llf2} := w_{llf.0.eq} \cdot \left[(L_f) \cdot (W_f)\right]$

$Wt_{f.mx} = (wgt_{flr} + w_{misc.f}) \cdot (L_f) \cdot (W_f)$ $\quad Wt_{f.mn} = (wgt_{flr}) \cdot (L_f) \cdot (W_f)$

$Wt_{f.mx}^T = (5.6 \quad 6.5 \quad 7.3 \quad 8.2 \quad 9.1 \quad 10 \quad 10.9 \quad 11.6 \quad 12.5) \cdot kip$ Max Floor Weight

$Wt_{f.mn}^T = (5.1 \quad 5.9 \quad 6.7 \quad 7.5 \quad 8.3 \quad 9.1 \quad 9.8 \quad 10.5 \quad 11.3) \cdot kip$ Min Floor Weight

$Wt_{25llf0}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf1}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.0} = 250 \cdot psf$

$Wt_{25llf2}^T = (7.7 \quad 9.1 \quad 10.5 \quad 11.8 \quad 13.2 \quad 14.6 \quad 16 \quad 17.3 \quad 18.7) \cdot kip$ 25% Floor, $w_{llf.1} = 200 \cdot psf$

$Wt_{EQ.f.f0} := Wt_{f.mx} + Wt_{25llf0}$ $\quad Wt_{EQ.f.f1} := Wt_{f.mx} + Wt_{25llf1}$ $\quad Wt_{EQ.f.f2} := Wt_{f.mx} + Wt_{25llf2}$

Seismic Weight of Floor, $w_{llf.0} = 250 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.1} = 200 \cdot psf$ $\quad$ Seismic Weight of Floor, $w_{llf.2} = 150 \cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$Wt_{EQ.f.f0}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.0} = 250\cdot psf$

$Wt_{EQ.f.f1}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.1} = 200\cdot psf$

$Wt_{EQ.f.f2}^{T} = (13.3 \quad 15.6 \quad 17.8 \quad 20.1 \quad 22.3 \quad 24.6 \quad 26.8 \quad 28.9 \quad 31.2)\cdot kip$ Floor Seismic Weight, $w_{llf.2} = 150\cdot psf$

$W_{lat} = 55.64\cdot psf$ Combined Windard & Leeward Pressure

$V_{w.Y.h0} := W_{lat}\cdot H_{E_0}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1} := W_{lat}\cdot H_{E_1}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2} := W_{lat}\cdot H_{E_2}\cdot L_f$ Wind on SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.X.h0} := W_{lat}\cdot H_{E_0}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} := W_{lat}\cdot H_{E_1}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} := W_{lat}\cdot H_{E_2}\cdot W_{ew}$ Wind on EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.Y.h0}^{T} = (6 \quad 7 \quad 8.1 \quad 9.1 \quad 10.2 \quad 11.2 \quad 12.3 \quad 13.3 \quad 14.4)\cdot kip$ Wind SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.Y.h1}^{T} = (6.6 \quad 7.8 \quad 8.9 \quad 10.1 \quad 11.2 \quad 12.4 \quad 13.6 \quad 14.7 \quad 15.9)\cdot kip$ Wind SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.Y.h2}^{T} = (7 \quad 8.2 \quad 9.5 \quad 10.7 \quad 12 \quad 13.2 \quad 14.4 \quad 15.7 \quad 16.9)\cdot kip$ Wind SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.X.h0} = 6.15\cdot kip$ Wind EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.X.h1} = 6.8\cdot kip$ Wind EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.X.h2} = 7.24\cdot kip$ Wind EW (X-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.Y.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot L_f$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.X.h0} := W_{lat}\cdot\frac{H_{E_0}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} := W_{lat}\cdot\frac{H_{E_1}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} := W_{lat}\cdot\frac{H_{E_2}}{2}\cdot W_{ew}$ Wind on Roof Diapragm EW (X-Direction) $H_{wall_2} = 10.7\,ft$

$V_{w.r.Y.h0}^{T} = (3 \quad 3.5 \quad 4 \quad 4.6 \quad 5.1 \quad 5.6 \quad 6.1 \quad 6.7 \quad 7.2)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_0} = 9\,ft$

$V_{w.r.Y.h1}^{T} = (3.3 \quad 3.9 \quad 4.5 \quad 5 \quad 5.6 \quad 6.2 \quad 6.8 \quad 7.4 \quad 8)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_1} = 10\,ft$

$V_{w.r.Y.h2}^{T} = (3.5 \quad 4.1 \quad 4.7 \quad 5.4 \quad 6 \quad 6.6 \quad 7.2 \quad 7.8 \quad 8.5)\cdot kip$ Wind on Roof Diapragm SW (Y-Direction) $H_{wall_2} = 10.67\,ft$

$V_{w.r.X.h0} = 3.08\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_0} = 9\,ft$

$V_{w.r.X.h1} = 3.4\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_1} = 10\,ft$

$V_{w.r.X.h2} = 3.62\cdot kip$ Wind RDiap. EW (X-Dir.) $H_{wall_2} = 10.67\,ft$

Seismic Weights & Shears

$C_s = 0.24$ $C_{s.ref}$ = "Seismic Response Coeff., Eqn (12.8-2)"

$Wt_{EQ.bh0f0}^{T} = (41 \quad 47 \quad 52 \quad 57 \quad 63 \quad 68 \quad 74 \quad 79 \quad 84)\cdot kip$ EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$V_{EQ.bh0f0} := C_s\cdot Wt_{EQ.bh0f1}$ $H_{wall_0} = 9\,ft$, $w_{llf.0} = 250\cdot psf$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH LV
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$V_{EQ.bh0f0}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh1f0}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh1f0} := C_s \cdot Wt_{EQ.bh1f0}$

$V_{EQ.bh1f0}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh2f0}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.0} = 250\cdot psf$

$V_{EQ.bh2f0} := C_s \cdot Wt_{EQ.bh2f0}$

$V_{EQ.bh2f0}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh0f1}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh0f1} := C_s \cdot Wt_{EQ.bh0f1}$

$V_{EQ.bh0f1}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh1f1}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh1f1} := C_s \cdot Wt_{EQ.bh1f1}$

$V_{EQ.bh1f1}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh2f1}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.1} = 200\cdot psf$

$V_{EQ.bh2f1} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f1}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh0f2}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_0} = 9\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh0f2} := C_s \cdot Wt_{EQ.bh0f2}$

$V_{EQ.bh0f2}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh1f2}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_1} = 10\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh1f2} := C_s \cdot Wt_{EQ.bh1f2}$

$V_{EQ.bh1f2}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.bh2f2}^{T} = (41 \;\; 47 \;\; 52 \;\; 57 \;\; 63 \;\; 68 \;\; 74 \;\; 79 \;\; 84)\cdot kip$

EQ Wt/Base Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$, $w_{llf.2} = 150\cdot psf$

$V_{EQ.bh2f2} := C_s \cdot Wt_{EQ.bh2f2}$

$V_{EQ.bh2f2}^{T} = (10 \;\; 11 \;\; 12 \;\; 14 \;\; 15 \;\; 16 \;\; 18 \;\; 19 \;\; 20)\cdot kip$

$Wt_{EQ.rh0}^{T} = (19 \;\; 21 \;\; 23 \;\; 26 \;\; 28 \;\; 30 \;\; 33 \;\; 35 \;\; 37)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 35\cdot psf$ $H_{wall_0} = 9\,ft$

$V_{EQ.rh0} := C_s \cdot Wt_{EQ.rh0}$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$V_{EQ.rh0}^T = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

$Wt_{EQ.rh1}^T = (19 \;\; 21 \;\; 23 \;\; 26 \;\; 28 \;\; 30 \;\; 33 \;\; 35 \;\; 37)\cdot kip$

Roof Diap. Wt/Shear

$w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

$V_{EQ.rh1} := C_s \cdot Wt_{EQ.rh1}$

$V_{EQ.rh1}^T = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

$Wt_{EQ.rh2}^T = (19 \;\; 21 \;\; 23 \;\; 26 \;\; 28 \;\; 30 \;\; 33 \;\; 35 \;\; 37)\cdot kip$

Roof Diap. Wt/Shear $w_{llr} = 35\cdot psf$

$H_{wall_2} = 10.7\,ft$

$V_{EQ.rh2} := C_s \cdot Wt_{EQ.rh2}$

$V_{EQ.rh2}^T = (4 \;\; 5 \;\; 6 \;\; 6 \;\; 7 \;\; 7 \;\; 8 \;\; 8 \;\; 9)\cdot kip$

b. Roof Diaphragm

$w_{EQh0.y} := \frac{V_{EQ.rh0}}{L_r}$ $\quad w_{EQh0.y}^T = (362 \;\; 350 \;\; 341 \;\; 335 \;\; 329 \;\; 324 \;\; 321 \;\; 317 \;\; 315)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.y} := \frac{V_{EQ.rh1}}{L_r}$ $\quad w_{EQh1.y}^T = (362 \;\; 350 \;\; 341 \;\; 335 \;\; 329 \;\; 324 \;\; 321 \;\; 317 \;\; 315)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.y} := \frac{V_{EQ.rh2}}{L_r}$ $\quad w_{EQh2.y}^T = (362 \;\; 350 \;\; 341 \;\; 335 \;\; 329 \;\; 324 \;\; 321 \;\; 317 \;\; 315)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{EQh0.x} := \frac{V_{EQ.rh0}}{W_r}$ $\quad w_{EQh0.x}^T = (372 \;\; 419 \;\; 465 \;\; 511 \;\; 557 \;\; 604 \;\; 650 \;\; 696 \;\; 743)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_0} = 9\,ft$

$w_{EQh1.x} := \frac{V_{EQ.rh1}}{W_r}$ $\quad w_{EQh1.x}^T = (372 \;\; 419 \;\; 465 \;\; 511 \;\; 557 \;\; 604 \;\; 650 \;\; 696 \;\; 743)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_1} = 10\,ft$

$w_{EQh2.x} := \frac{V_{EQ.rh2}}{W_r}$ $\quad w_{EQh2.x}^T = (372 \;\; 419 \;\; 465 \;\; 511 \;\; 557 \;\; 604 \;\; 650 \;\; 696 \;\; 743)\cdot plf$

R Diap. EQ Dist. Load $w_{llr} = 35\cdot psf \quad H_{wall_2} = 10.67\,ft$

$w_{W.h0.y} := \frac{V_{w.r.Y.h0}}{L_r}$ $\quad w_{W.h0.y}^T = (241 \;\; 245 \;\; 247 \;\; 249 \;\; 250 \;\; 251 \;\; 252 \;\; 253 \;\; 254)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_0} = 9\,ft$

$w_{W.h1.y} := \frac{V_{w.r.Y.h1}}{L_r}$ $\quad w_{W.h1.y}^T = (267 \;\; 270 \;\; 273 \;\; 275 \;\; 277 \;\; 278 \;\; 279 \;\; 280 \;\; 281)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_1} = 10\,ft$

$w_{W.h2.y} := \frac{V_{w.r.Y.h2}}{L_r}$ $\quad w_{W.h2.y}^T = (284 \;\; 288 \;\; 290 \;\; 292 \;\; 294 \;\; 296 \;\; 297 \;\; 298 \;\; 299)\cdot plf$

R Diap. Wind Dist. Load $H_{wall_2} = 10.67\,ft$

$w_{W.h0.x} := \frac{V_{w.r.X.h0}}{W_r}$

$w_{W.h0.x} = 256.4\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_0} = 9\,ft$

$w_{W.h1.x} := \frac{V_{w.r.X.h1}}{W_r}$

$w_{W.h1.x} = 283.4\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_1} = 10\,ft$

$w_{W.h2.x} := \frac{V_{w.r.X.h2}}{W_r}$

$w_{W.h2.x} = 301.5\cdot plf$

R Diap. Wind Dist. Load

$H_{wall_2} = 10.7\,ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$M_{u.bh0.y}^{T} = (6.9 \;\; 9 \;\; 11.4 \;\; 14.1 \;\; 17 \;\; 20.2 \;\; 23.7 \;\; 27.5 \;\; 31.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.y}^{T} = (6.9 \;\; 9 \;\; 11.4 \;\; 14.1 \;\; 17 \;\; 20.2 \;\; 23.7 \;\; 27.5 \;\; 31.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.y}^{T} = (6.9 \;\; 9 \;\; 11.4 \;\; 14.1 \;\; 17 \;\; 20.2 \;\; 23.7 \;\; 27.5 \;\; 31.6)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along SW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.y}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along SW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.bh0.x}^{T} = (6.7 \;\; 7.5 \;\; 8.4 \;\; 9.2 \;\; 10 \;\; 10.9 \;\; 11.7 \;\; 12.5 \;\; 13.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$M_{u.bh1.x}^{T} = (6.7 \;\; 7.5 \;\; 8.4 \;\; 9.2 \;\; 10 \;\; 10.9 \;\; 11.7 \;\; 12.5 \;\; 13.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$M_{u.bh2.x}^{T} = (6.7 \;\; 7.5 \;\; 8.4 \;\; 9.2 \;\; 10 \;\; 10.9 \;\; 11.7 \;\; 12.5 \;\; 13.4)\cdot kip\cdot ft$

Roof Diaphragm EQ Moment, along EW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$V_{u.bh0.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_0} = 9\,ft$

$V_{u.bh1.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_1} = 10\,ft$

$V_{u.bh2.x}^{T} = (2.2 \;\; 2.5 \;\; 2.8 \;\; 3.1 \;\; 3.3 \;\; 3.6 \;\; 3.9 \;\; 4.2 \;\; 4.5)\cdot kip$

Roof Diaphragm EQ Shear along EW $w_{llr} = 35\cdot psf$, $H_{wall_2} = 10.7ft$

$M_{u.b.h0.y}^{T} = (4.6 \;\; 6.3 \;\; 8.2 \;\; 10.5 \;\; 12.9 \;\; 15.7 \;\; 18.7 \;\; 22 \;\; 25.5)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.y}^{T} = (5.1 \;\; 6.9 \;\; 9.1 \;\; 11.6 \;\; 14.3 \;\; 17.3 \;\; 20.7 \;\; 24.3 \;\; 28.2)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.y}^{T} = (5.4 \;\; 7.4 \;\; 9.7 \;\; 12.3 \;\; 15.2 \;\; 18.4 \;\; 22 \;\; 25.8 \;\; 30)\cdot kip\cdot ft$

Roof Diaphragm Wind Moment, along SW - $H_{wall_2} = 10.7ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.co

$V_{u.b.h0.y}^{T} = (1.5 \quad 1.8 \quad 2 \quad 2.3 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_0} = 9\,ft$

$V_{u.b.h1.y}^{T} = (1.6 \quad 1.9 \quad 2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.4 \quad 3.7 \quad 4)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_1} = 10\,ft$

$V_{u.b.h2.y}^{T} = (1.8 \quad 2.1 \quad 2.4 \quad 2.7 \quad 3 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2)\cdot kip$

Roof Diaphragm Wind Shear along SW - $H_{wall_2} = 10.7 ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{u.b.h0.x} = 4.6\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$M_{u.b.h1.x} = 5.1\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$M_{u.b.h2.x} = 5.4\cdot kip\cdot ft$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7 ft$

$v_{u.b.h0.x} = 1.5\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_0} = 9\,ft$

$v_{u.b.h1.x} = 1.7\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_1} = 10\,ft$

$v_{u.b.h2.x} = 1.8\cdot kip$ Roof Diaphragm Wind Moment, along EW - $H_{wall_2} = 10.7 ft$

Roof Perimeter Reinforcement - Design along SW

$B_{7.1} = \text{"\#6"}$ $B_{7.1.ref} = \text{"Roof Perim. Reinf."}$ $N_{B.7.1} = 1$ $N_{B.7.1.ref} = \text{"\# Roof Perim. Bars"}$

$D_{7b.1} = 0.8\cdot in$ $D_{7b.1.ref} = \text{"Roof Perim. Reiinf. Bar Diam."}$ $A_{7b.1} = 0.4\cdot in^2$ $A_{7b.1.ref} = \text{"Roof Perim. Reinf. Area"}$

$As_1 := As_{2.2} + A_{7b.1}$ $As_1 = 0.64\cdot in^2$ $As_{1.mn.ref} := \text{"Roof Diaph.Reinf."}$ $\rho_{b.1} := \frac{As_1}{W_r\cdot t_{r.e}}$ $\rho_{b.1} = 0.00111$

$d_{b.1} := W_r - 0.75in - \frac{D_{7b.1}}{2}$ $d_{b.1} = 11.9\,ft$ Depth of Rebar $a_{b.1} := \frac{As_1\cdot f_y}{0.85\cdot f'_c\cdot t_{r.e}}$ $a_{b.1} = 2.51\cdot in$ Depth of Compr. Block

$M_{b.1} := A_{7b.1}\cdot f_y\cdot\left(d_{b.1} - \frac{a_{b.1}}{2}\right) + As_{2.2}\cdot f_y\cdot\left(\frac{d_{b.1}}{2} - \frac{a_{b.1}}{2}\right)$ $M_{b.1} = 381.7\cdot kip\cdot ft$ Nom. Moment Capacity of Roof Diaphragm

$c_{b.1} := \frac{a_{b.1}}{0.85}$ $c_{b.1} = 3\cdot in$ Dist. from Comp Face to NA $\varepsilon_{t.b.1} := 0.003\cdot\frac{d_{b.1} - c_{b.1}}{c_{b.1}}$ $\varepsilon_{t.b.1} = 0.14216$ Strain in Ten. Rebar

$\phi_{b.1} := \text{if}\left[\varepsilon_{t.b.1} \geq 0.005, 0.9, \text{if}\left[\varepsilon_{t.b.1} \leq 0.002, 0.65, 0.5 + (\varepsilon_{t.b.1} - 0.002)\cdot 50\right]\right]$ $\phi_{b.1} = 0.9$ Flexural Reduction Factor

$\phi M_{b.1} := \phi_{b.1}\cdot M_{b.1}$ $\phi M_{b.1} = 343.6\cdot kip\cdot ft$ Factored Moment Capacity of Roof Diaphragm

$Unity_{b.1a} = \left(\frac{M_{u.bh0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.bh1.y}}{\phi M_{b.1_1}} \quad \frac{M_{u.bh2.y}}{\phi M_{b.1_2}}\right)$ EQ Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35\cdot psf$

$Unity_{b.1a}^{T} = \begin{pmatrix} 2 & 2.6 & 3.3 & 4.1 & 4.9 & 5.9 & 6.9 & 8 & 9.2 \\ 2 & 2.6 & 3.3 & 4.1 & 4.9 & 5.9 & 6.9 & 8 & 9.2 \\ 2 & 2.6 & 3.3 & 4.1 & 4.9 & 5.9 & 6.9 & 8 & 9.2 \end{pmatrix}\cdot\%$

$chk_{b.1a} := \text{if}\left(\max\left(Unity_{b.1a}\right) < 1, \text{"OK"}, \text{"NG"}\right)$

$chk_{b.1a} = \text{"OK"}$

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$$Unity_{b.1b} = \left(\frac{M_{u.b.h0.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h1.y}}{\phi M_{b.1_0}} \quad \frac{M_{u.b.h2.y}}{\phi M_{b.1_0}} \right)$$

Wind Roof along SW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.1b}^T = \begin{pmatrix} 1.3 & 1.8 & 2.4 & 3 & 3.8 & 4.6 & 5.4 & 6.4 & 7.4 \\ 1.5 & 2 & 2.6 & 3.4 & 4.2 & 5 & 6 & 7.1 & 8.2 \\ 1.6 & 2.1 & 2.8 & 3.6 & 4.4 & 5.4 & 6.4 & 7.5 & 8.7 \end{pmatrix} \cdot \%$$

$chk_{b.1b} := if\left(max\left(Unity_{b.1b}\right) < 1, "OK", "NG"\right)$

$chk_{b.1b} = "OK"$

Roof Perimeter Reinforcement - Design along EW

$As_2 := As_{2.1} + A_{7b.1}$ $\quad As_2 = 0.68 \cdot in^2$ $\quad As_{2.mn.ref} := "Roof Diaph.Reinf."$ $\quad d_{b.2} := L_r - 0.5in - \frac{D_{7b.1}}{2}$ Depth of Rebar

$a_{b.2} := \frac{As_2 \cdot f_y}{0.85 \cdot f'_c \cdot t_{r.e}}$ $\quad a_{b.1} = 2.51 \cdot in$ Depth of Compr. Block $\quad \rho_{b.2} := \frac{As_2}{L_r \cdot t_{r.e}}$

$d_{b.2}^T = (12.26 \quad 14.26 \quad 16.26 \quad 18.26 \quad 20.26 \quad 22.26 \quad 24.26 \quad 26.26 \quad 28.26)\,ft$

$\rho_{b.2}^T = (0.00115 \quad 0.00099 \quad 0.00087 \quad 0.00077 \quad 0.0007 \quad 0.00063 \quad 0.00058 \quad 0.00054 \quad 0.0005)$

$$M_{b.2a} := A_{7b.1} \cdot f_y \cdot \left(d_{b.2} - \frac{a_{b.2}}{2} \right) + As_{2.1} \cdot f_y \cdot \left(\frac{d_{b.2}}{2} - \frac{a_{b.2}}{2} \right)$$

$M_{b.2a}^T = (408 \quad 475 \quad 542 \quad 609 \quad 676 \quad 744 \quad 811 \quad 878 \quad 945) \cdot kip \cdot ft$ Nom. Moment Capacity, Roof Diaphragm

$c_{b.2} := \frac{a_{b.2}}{0.85}$ $\quad c_{b.2} = 3.1 \cdot in$ Dist. from Comp Face to NA $\quad \varepsilon_{t.b.2a} := 0.003 \cdot \frac{d_{b.2} - c_{b.2}}{c_{b.2}}$

$\varepsilon_{t.b.2a}^T = (0.1377 \quad 0.1606 \quad 0.1836 \quad 0.2065 \quad 0.2295 \quad 0.2524 \quad 0.2754 \quad 0.2983 \quad 0.3213)$ Strain in Rebar

$\phi_{b.2} := if\left[\min\left(\varepsilon_{t.b.2a}\right) \geq 0.005, 0.9, if\left[\min\left(\varepsilon_{t.b.2a}\right) \leq 0.002, 0.65, 0.5 + \left(\min\left(\varepsilon_{t.b.2a}\right) - 0.002\right) \cdot 50\right]\right]$ $\quad \phi_{b.2} = 0.9$

Flexural Reduction Factor

$\phi M_{b.2a} := \phi_{b.1} \cdot M_{b.2a}$ $\quad \phi M_{b.2a}^T = (367 \quad 427 \quad 488 \quad 548 \quad 609 \quad 669 \quad 730 \quad 790 \quad 851) \cdot kip \cdot ft$

Factored Moment Capacity of Roof Diaphragm

$$Unity_{b.2a} = \left(\frac{M_{u.bh0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.bh1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.bh2.x}}{\phi M_{b.2a_2}} \right)$$

EQ Roof Diaph. along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.2a}^T = \begin{pmatrix} 1.8 & 1.8 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.8 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 & 1.6 \\ 1.8 & 1.8 & 1.7 & 1.7 & 1.6 & 1.6 & 1.6 & 1.6 & 1.6 \end{pmatrix} \cdot \%$$

$chk_{b.2a} := if\left(max\left(Unity_{b.2a}\right) < 1, "OK", "NG"\right)$

$chk_{b.2a} = "OK"$

$$Unity_{b.2b} = \left(\frac{M_{u.b.h0.x}}{\phi M_{b.2a_0}} \quad \frac{M_{u.b.h1.x}}{\phi M_{b.2a_1}} \quad \frac{M_{u.b.h2.x}}{\phi M_{b.2a_2}} \right)$$

Wind Roof along EW $H_{wall}^T = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.2b}^T = \begin{pmatrix} 1.3 & 1.1 & 0.9 & 0.8 & 0.8 & 0.7 & 0.6 & 0.6 & 0.5 \\ 1.4 & 1.2 & 1 & 0.9 & 0.8 & 0.8 & 0.7 & 0.6 & 0.6 \\ 1.5 & 1.3 & 1.1 & 1 & 0.9 & 0.8 & 0.7 & 0.7 & 0.6 \end{pmatrix} \cdot \%$$

$chk_{b.2b} := if\left(max\left(Unity_{b.2b}\right) < 1, "OK", "NG"\right)$

$chk_{b.2b} = "OK"$

Shear

$A_{cv.b.3a.1} := t_{r.e} \cdot W_r$ $\quad A_{cv.b.3a.1} = 576 \cdot in^2$ Roof Shear Area for $t_{r.e} = 4 \cdot in$ & $W_r = 12 \cdot ft$ $\quad \rho_{b.1} = 0.00111$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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$$A_{cv.b.3b.1} := \begin{pmatrix} t_{r.e}\cdot L_{r_0} \\ t_{r.e}\cdot L_{r_1} \\ t_{r.e}\cdot L_{r_2} \\ t_{r.e}\cdot L_{r_3} \\ t_{r.e}\cdot L_{r_4} \\ t_{r.e}\cdot L_{r_5} \\ t_{r.e}\cdot L_{r_6} \\ t_{r.e}\cdot L_{r_7} \\ t_{r.e}\cdot L_{r_8} \end{pmatrix} \qquad A_{cv.b.3b.1} = \begin{pmatrix} 592 \\ 688 \\ 784 \\ 880 \\ 976 \\ 1072 \\ 1168 \\ 1264 \\ 1360 \end{pmatrix}\cdot in^2 \qquad \text{Roof Shear Area for } t_{r.e} = 4\cdot in \text{ \& } L_r = \begin{pmatrix} 1776 \\ 2064 \\ 2352 \\ 2640 \\ 2928 \\ 3216 \\ 3504 \\ 3792 \\ 4080 \end{pmatrix}\cdot\frac{in^2}{ft}$$

$$\rho_{b.2}^{T} = (0.00115 \quad 0.00099 \quad 0.00087 \quad 0.00077 \quad 0.0007 \quad 0.00063 \quad 0.00058 \quad 0.00054 \quad 0.0005)$$

Per ACI Nominal Shear shall not exceed: $V_{max} = A_{cv}\cdot\left(2\cdot\sqrt{f'_c} + \rho\cdot f_y\right)$ $V_{b.3a.1.mx} = 115.7\cdot kip$ Max Allowable for $t_{r.e} = 4\cdot in$ & $W_r = 12\cdot ft$

$$V_{b.3a.2.mx}^{T} = (120.2 \quad 133.1 \quad 146 \quad 158.9 \quad 171.7 \quad 184.6 \quad 197.5 \quad 210.4 \quad 223.3)\cdot kip$$

Max Allowable Shear for $t_{r.e} = 4\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$V_{b.3b.1} = 32.8\cdot kip$ Nominal Shear Strength for $t_{r.e} = 4\cdot in$ & $W_r = 12\cdot ft$

$\phi V_c = \min\left(V_{b.3a.1.mx}, V_{b.3b.1}\right)$

Roof Diaph. Shear Strength for $t_{r.e} = 4\cdot in$ & $W_r = 12\cdot ft$ $\phi V_{c.b.3a.1} = 24.6\cdot kip$

$$V_{b.3b.2}^{T} = (33.8 \quad 39.2 \quad 44.7 \quad 50.2 \quad 55.7 \quad 61.1 \quad 66.6 \quad 72.1 \quad 77.5)\cdot kip$$

Nominal Shear Strength for $t_{r.e} = 4\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

$\phi V_c = \min\left(V_{b.3a.2.mx}, V_{b.3b.2}\right)$

$$\phi V_{c.b.3a.2}^{T} = (25.3 \quad 29.4 \quad 33.5 \quad 37.6 \quad 41.7 \quad 45.8 \quad 49.9 \quad 54.1 \quad 58.2)\cdot kip$$

Roof Diaph. Shear Strength for $t_{r.e} = 4\cdot in$ & $L_r^{T} = (12.3 \quad 14.3 \quad 16.3 \quad 18.3 \quad 20.3 \quad 22.3 \quad 24.3 \quad 26.3 \quad 28.3)\cdot ft$

Seismic Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35\cdot psf$

$$Unity_{b.3a}^{T} = \begin{pmatrix} 9.1 & 10.2 & 11.3 & 12.4 & 13.6 & 14.7 & 15.8 & 17 & 18.1 \\ 9.1 & 10.2 & 11.3 & 12.4 & 13.6 & 14.7 & 15.8 & 17 & 18.1 \\ 9.1 & 10.2 & 11.3 & 12.4 & 13.6 & 14.7 & 15.8 & 17 & 18.1 \end{pmatrix}\cdot\%$$

$chk_{b.3a} = "OK"$

Wind Roof Diaph. along SW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35\cdot psf$

$$Unity_{b.3d}^{T} = \begin{pmatrix} 6 & 7.1 & 8.2 & 9.3 & 10.3 & 11.4 & 12.5 & 13.5 & 14.6 \\ 6.7 & 7.9 & 9 & 10.2 & 11.4 & 12.6 & 13.8 & 15 & 16.2 \\ 7.1 & 8.4 & 9.6 & 10.9 & 12.1 & 13.4 & 14.7 & 15.9 & 17.2 \end{pmatrix}\cdot\%$$

$chk_{b.3d} = "OK"$

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APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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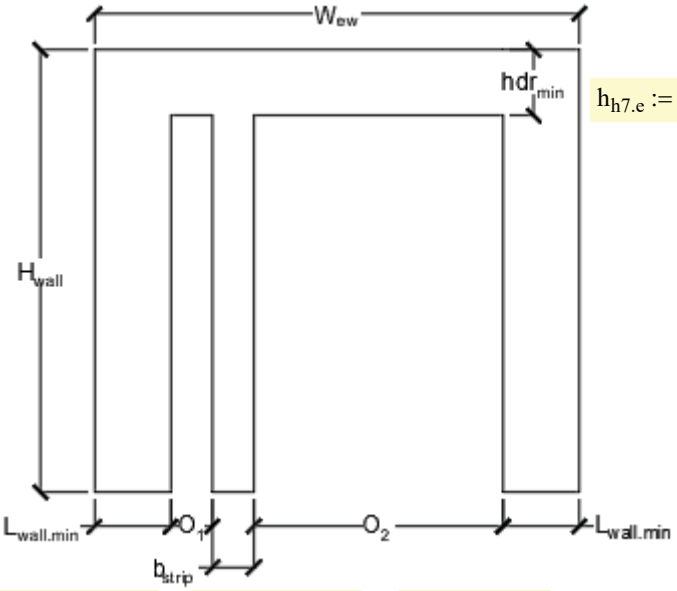
Seismic Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

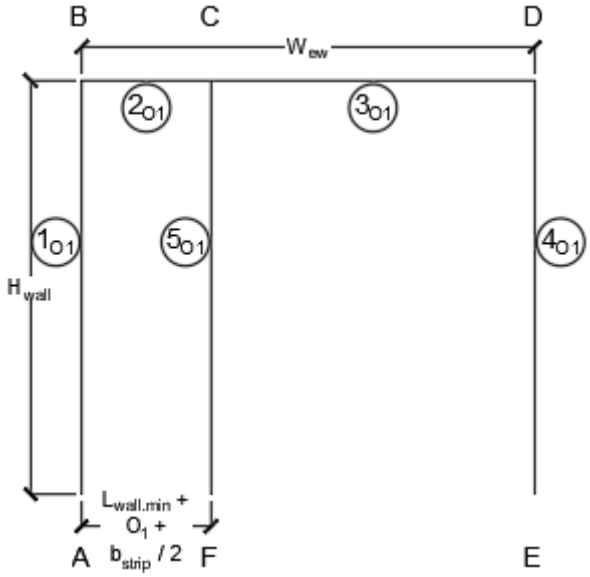
$Unity_{b.3g}^{T} = (8.8 \quad 8.5 \quad 8.3 \quad 8.1 \quad 8 \quad 7.9 \quad 7.8 \quad 7.7 \quad 7.7) \cdot \%$ $chk_{b.3j} = "OK"$

Wind Roof Diaph. along EW $H_{wall}^{T} = (9 \quad 10 \quad 10.7)\,ft$ & $w_{llr} = 35 \cdot psf$

$$Unity_{b.3j}^{T} = \begin{pmatrix} 6.1 & 5.2 & 4.6 & 4.1 & 3.7 & 3.4 & 3.1 & 2.8 & 2.6 \\ 6.7 & 5.8 & 5.1 & 4.5 & 4.1 & 3.7 & 3.4 & 3.1 & 2.9 \\ 7.1 & 6.1 & 5.4 & 4.8 & 4.3 & 3.9 & 3.6 & 3.3 & 3.1 \end{pmatrix} \cdot \%$$ $chk_{b.3j} = "OK"$

END WALL DESIGN





$L_{ew.mn} := 22in$ $b_{strip.O1} := 26in$ $W1_{O1} := 12in$ $W2_{O1} := W_{ew} - 2 \cdot L_{ew.mn} - W1_{O1} - b_{strip.O1}$ $W2_{O1} = 58 \cdot in$

$$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix} ft \qquad H_{wall} = \begin{pmatrix} 108 \\ 120 \\ 128 \end{pmatrix} \cdot in$$

$E_c = 2837.5 \cdot ksi$ $E_{c.ref} = "Modulus of Elas. Conc."$

Seismic Lateral Loads to Endwall

$$Veq_{ewho} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h0}}{2} \right) \qquad Veq_{ewho}^{T} = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5) \cdot kip$$

Height $H_{wall_0} = 9\,ft$

$$Veq_{ewh1} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h1}}{2} \right) \qquad Veq_{ewh1}^{T} = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5) \cdot kip$$

Height $H_{wall_1} = 10\,ft$

$$Veq_{ewh2} := \frac{C_s}{2} \cdot \left(Wt_{rf.mx} + Wt_{25ll} + \frac{Wt_{EQ.w.h2}}{2} \right) \qquad Veq_{ewh2}^{T} = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5) \cdot kip$$

Height $H_{wall_2} = 10.7\,ft$

Joint A

$DF_{A.O1} := 0$ $FEM_{A.O1} := FEA_{Guess}$

Joint F

$DF_{F.O1} := 0$ $FEM_{F.O1} := FEA_{Guess}$

Joint E

$DF_{E.O1} := 0$ $FEM_{E.O1} := FEA_{Guess}$

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Const. Type: VB
Occupancy: S2
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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Moment Distribution For Wall Height $H_{wall_0} = 9\,ft$

Joint B

$K_{B.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{B.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h0} := \sum K_{B.O1.h0}$ $K_{BO1.Tot.h0} = 71 \cdot in^3$

$DF_{BA.O1.h0} := \frac{K_{B.O1.h0_0}}{K_{BO1.Tot.h0}}$ $DF_{BA.O1.h0} = 0.463$

$DF_{BC.O1.h0} := \frac{K_{B.O1.h0_1}}{K_{BO1.Tot.h0}}$ $DF_{BC.O1.h0} = 0.537$

Joint D

$K_{D.O1.h0} := \begin{pmatrix} K_{M1O1.h0} \\ K_{M2O1.h0} \end{pmatrix}$ $K_{D.O1.h0} = \begin{pmatrix} 32.9 \\ 38.1 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h0} := \sum K_{D.O1.h0}$ $K_{DO1.Tot.h0} = 71 \cdot in^3$

$DF_{DC.O1.h0} := \frac{K_{D.O1.h0_0}}{K_{DO1.Tot.h0}}$ $DF_{DC.O1.h0} = 0.463$

$DF_{DE.O1.h0} := \frac{K_{D.O1.h0_1}}{K_{DO1.Tot.h0}}$ $DF_{DE.O1.h0} = 0.537$

Joint C

$K_{C.O1.h0} := \begin{pmatrix} K_{M2O1.h0} \\ K_{M3O1.h0} \\ K_{M5O1.h0} \end{pmatrix}$ $K_{C.O1.h0} = \begin{pmatrix} 38.1 \\ 19.4 \\ 54.2 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h0} := \sum K_{C.O1.h0}$ $K_{CO1.Tot.h0} = 111.7 \cdot in^3$

$DF_{CB.O1.h0} := \frac{K_{C.O1.h0_0}}{K_{CO1.Tot.h0}}$ $DF_{CB.O1.h0} = 0.341$

$DF_{CD.O1.h0} := \frac{K_{C.O1.h0_1}}{K_{CO1.Tot.h0}}$ $DF_{CD.O1.h0} = 0.173$

$DF_{CF.O1.h0} := \frac{K_{C.O1.h0_2}}{K_{CO1.Tot.h0}}$ $DF_{CF.O1.h0} = 0.486$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No.	
of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$M_{L.ewh0}^T = (3.8 \;\; 4.2 \;\; 4.7 \;\; 5.2 \;\; 5.6 \;\; 6.1 \;\; 6.6 \;\; 7.1 \;\; 7.5) \cdot kip \cdot ft$

$V_{L.ewh0}^T = (0.7 \;\; 0.8 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.4) \cdot kip$

$M_{hdrh0}^T = (2.8 \;\; 3.1 \;\; 3.5 \;\; 3.8 \;\; 4.1 \;\; 4.5 \;\; 4.8 \;\; 5.2 \;\; 5.5) \cdot kip \cdot ft$

$V_{hdrh0}^T = (1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.6 \;\; 1.7 \;\; 1.8 \;\; 1.9) \cdot kip$

$M_{b.striph0}^T = (4.2 \;\; 4.7 \;\; 5.2 \;\; 5.7 \;\; 6.2 \;\; 6.7 \;\; 7.3 \;\; 7.8 \;\; 8.3) \cdot kip \cdot ft$

$V_{b.striph0}^T = (0.9 \;\; 1 \;\; 1.1 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.1 \;\; 1.2 \;\; 1.3) \cdot kip$

Moment Distribution For Wall Height $H_{wall_1} = 10\,ft$

Joint B

$K_{B.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{B.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h1} := \sum K_{B.O1.h1}$ $K_{BO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{BA.O1.h1} := \frac{K_{B.O1.h1_0}}{K_{BO1.Tot.h1}}$ $DF_{BA.O1.h1} = 0.378$

$DF_{BC.O1.h1} := \frac{K_{B.O1.h1_1}}{K_{BO1.Tot.h1}}$ $DF_{BC.O1.h1} = 0.622$

Joint C

$K_{C.O1.h1} := \begin{pmatrix} K_{M2O1.h1} \\ K_{M3O1.h1} \\ K_{M5O1.h1} \end{pmatrix}$ $K_{C.O1.h1} = \begin{pmatrix} 48.6 \\ 24.6 \\ 48.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h1} := \sum K_{C.O1.h1}$ $K_{CO1.Tot.h1} = 122.1 \cdot in^3$

$DF_{CB.O1.h1} := \frac{K_{C.O1.h1_0}}{K_{CO1.Tot.h1}}$ $DF_{CB.O1.h1} = 0.399$

$DF_{CD.O1.h1} := \frac{K_{C.O1.h1_1}}{K_{CO1.Tot.h1}}$ $DF_{CD.O1.h1} = 0.201$

Joint D

$K_{D.O1.h1} := \begin{pmatrix} K_{M1O1.h1} \\ K_{M2O1.h1} \end{pmatrix}$ $K_{D.O1.h1} = \begin{pmatrix} 29.6 \\ 48.6 \end{pmatrix} \cdot in^3$ $DF_{CF.O1.h1} := \frac{K_{C.O1.h1_2}}{K_{CO1.Tot.h1}}$ $DF_{CF.O1.h1} = 0.4$

$K_{DO1.Tot.h1} := \sum K_{D.O1.h1}$ $K_{DO1.Tot.h1} = 78.2 \cdot in^3$

$DF_{DC.O1.h1} := \frac{K_{D.O1.h1_0}}{K_{DO1.Tot.h1}}$ $DF_{DC.O1.h1} = 0.378$

$DF_{DE.O1.h1} := \frac{K_{D.O1.h1_1}}{K_{DO1.Tot.h1}}$ $DF_{DE.O1.h1} = 0.622$

$M_{L.ewh1}{}^T = (4.3 \quad 4.9 \quad 5.4 \quad 5.9 \quad 6.5 \quad 7 \quad 7.6 \quad 8.1 \quad 8.6) \cdot kip \cdot ft$

$M_{hdrh1}{}^T = (3.5 \quad 3.9 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6 \quad 6.5 \quad 6.9) \cdot kip \cdot ft$

$M_{b.striph1}{}^T = (4.6 \quad 5.2 \quad 5.8 \quad 6.4 \quad 7 \quad 7.5 \quad 8.1 \quad 8.7 \quad 9.3) \cdot kip \cdot ft$

$V_{L.ewh1}{}^T = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$V_{hdrh1}{}^T = (1.5 \quad 1.7 \quad 1.9 \quad 2.1 \quad 2.3 \quad 2.5 \quad 2.7 \quad 2.9 \quad 3.1) \cdot kip$

$V_{b.striph1}{}^T = (0.9 \quad 1 \quad 1.1 \quad 0.8 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6 \quad 1.7) \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Moment Distribution For Wall Height $H_{wall_2} = 10.7ft$

Joint B

$K_{B.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{B.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{BO1.Tot.h2} := \sum K_{B.O1.h2}$ $K_{BO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{BA.O1.h2} := \frac{K_{B.O1.h2_0}}{K_{BO1.Tot.h2}}$ $DF_{BA.O1.h2} = 0.363$

$DF_{BC.O1.h2} := \frac{K_{B.O1.h2_1}}{K_{BO1.Tot.h2}}$ $DF_{BC.O1.h2} = 0.637$

Joint C

$K_{C.O1.h2} := \begin{pmatrix} K_{M2O1.h2} \\ K_{M3O1.h2} \\ K_{M5O1.h2} \end{pmatrix}$ $K_{C.O1.h2} = \begin{pmatrix} 48.6 \\ 24.6 \\ 45.8 \end{pmatrix} \cdot in^3$

$K_{CO1.Tot.h2} := \sum K_{C.O1.h2}$ $K_{CO1.Tot.h2} = 119 \cdot in^3$

$DF_{CB.O1.h2} := \frac{K_{C.O1.h2_0}}{K_{CO1.Tot.h2}}$ $DF_{CB.O1.h2} = 0.409$

$DF_{CD.O1.h2} := \frac{K_{C.O1.h2_1}}{K_{CO1.Tot.h2}}$ $DF_{CD.O1.h2} = 0.207$

$DF_{CF.O1.h2} := \frac{K_{C.O1.h2_2}}{K_{CO1.Tot.h2}}$ $DF_{CF.O1.h2} = 0.385$

Joint D

$K_{D.O1.h2} := \begin{pmatrix} K_{M1O1.h2} \\ K_{M2O1.h2} \end{pmatrix}$ $K_{D.O1.h2} = \begin{pmatrix} 27.7 \\ 48.6 \end{pmatrix} \cdot in^3$

$K_{DO1.Tot.h2} := \sum K_{D.O1.h2}$ $K_{DO1.Tot.h2} = 76.4 \cdot in^3$

$DF_{DC.O1.h2} := \frac{K_{D.O1.h2_0}}{K_{DO1.Tot.h2}}$ $DF_{DC.O1.h2} = 0.363$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DF_{DE.O1.h2} := \frac{K_{D.O1.h2_1}}{K_{DO1.Tot.h2}}$ $DF_{DE.O1.h2} = 0.637$

$M_{L.ewh2}^T = (4.6 \;\; 5.2 \;\; 5.8 \;\; 6.4 \;\; 7 \;\; 7.5 \;\; 8.1 \;\; 8.7 \;\; 9.3)\cdot kip\cdot ft$ $V_{L.ewh2}^T = (0.8 \;\; 0.9 \;\; 1 \;\; 1.1 \;\; 1.2 \;\; 1.3 \;\; 1.4 \;\; 1.5 \;\; 1.6)\cdot kip$

$M_{hdrh2}^T = (3.7 \;\; 4.2 \;\; 4.7 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.5 \;\; 7 \;\; 7.5)\cdot kip\cdot ft$ $V_{hdrh2}^T = (1.7 \;\; 1.9 \;\; 2.1 \;\; 2.3 \;\; 2.5 \;\; 2.7 \;\; 2.9 \;\; 3.1 \;\; 3.3)\cdot kip$

$M_{b.striph2}^T = (5 \;\; 5.6 \;\; 6.2 \;\; 6.8 \;\; 7.4 \;\; 8 \;\; 8.7 \;\; 9.3 \;\; 9.9)\cdot kip\cdot ft$ $V_{b.striph2}^T = (0.9 \;\; 1 \;\; 1.1 \;\; 0.8 \;\; 1.3 \;\; 1.4 \;\; 1.5 \;\; 1.6 \;\; 1.7)\cdot kip$

Reinforcement

$B_{3.1} = \text{"\#6"}$ $B_{3.1.ref} = \text{"Wall Vert. Perim. Reinf."}$ $N_{B.3.1} = 1$ $N_{B.3.1.ref} = \text{"\# Wall Vert. Perim. Bars"}$

$D_{3.1} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.1} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.2} = \text{"\#4"}$ $B_{3.2.ref} = \text{"Wall Top. Perim. Reinf."}$ $N_{B.3.2} = 2$ $N_{B.3.2.ref} = \text{"\# Wall Top Perim. Bars"}$

$D_{3.2} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.2} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.3} = \text{"\#6"}$ $B_{3.3.ref} = \text{"EW Bot Perim. Reinf."}$ $N_{B.3.3} = 1$ $N_{B.3.3.ref} = \text{"\# EW Bot Perim. Bars"}$

$D_{3.3} = 0.8\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.3} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.4} = \text{"\#4"}$ $B_{3.4.ref} = \text{"SW Bot Perim. Reinf."}$ $N_{B.3.4} = 2$ $N_{B.3.4.ref} = \text{"\# SW Bot Perim. Bars"}$

$D_{3.4} = 0.5\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.4} = 0.4\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$B_{3.5} = \text{"\#5"}$ $B_{3.5.ref} = \text{"Wall Opening Reinf."}$ $N_{B.3.5} = 2$ $N_{B.3.5.ref} = \text{"\# Wall Opening Bars"}$

$D_{3.5} = 0.6\cdot in$ $D_{ref} = \text{"Bar Diam."}$ $A_{3.5} = 0.6\cdot in^2$ $A_{ref} = \text{"Area of Reinf."}$

$d_{ewc} := L_{ew.mn} - 5in$ $d_{ewc} = 17\cdot in$ $d_{h7.e} := h_{h7.e} - 3.5in$ $d_{h7.e} = 15.5\cdot in$

$d_{strip} := b_{strip.O1} - 2.5in$ $d_{strip} = 23.5\cdot in$

$a_{ewc} := \frac{\min(A_{3.1}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{ewc} = 1.7\cdot in$ $a_{h.e} := \frac{\min(A_{3.2}, A_{3.5})\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{h.e} = 1.6\cdot in$

$a_{strip} := \frac{A_{3.5}\cdot f_y}{0.85\cdot f'_c\cdot t_w}$ $a_{strip} = 2.4\cdot in$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = \text{"Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"}$

$A_{mn.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot L_{ew.mn}$ $A_{mn.e} = 0.079\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.h.e} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.h.e} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$A_{mn.strip} := \rho_{t\_s.min}\cdot t_{slab}\cdot h_{h7.e}$ $A_{mn.strip} = 0.068\cdot in^2$ $a1_{TS.ref} = \text{"Temp/Shrink Min Area of Rebar, Tbl (24.4.3.2)"}$

$\rho_{ewc} := \frac{\min(A_{3.1}, A_{3.5})}{t_w\cdot d_{ewc}}$ $\rho_{ewc} = 0.0065$ $\rho_{h.e} := \frac{\min(A_{3.2}, A_{3.5})}{t_w\cdot d_{h7.e}}$ $\rho_{h.e} = 0.0065$

$\rho_{strip} := \frac{A_{3.5}}{t_w\cdot d_{ewc}}$ $\rho_{strip} = 0.0091$

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$chk_{c.1} := if\left(A_{mn.e} \le \min\left(A_{3.1}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.1} = "OK"$

$chk_{c.2} := if\left(A_{mn.e} \le \min\left(A_{3.2}, A_{3.5}\right), "OK", "NG"\right)$ $chk_{c.2} = "OK"$

$chk_{c.3} := if\left(A_{mn.strip} \le A_{3.5}, "OK", "NG"\right)$ $chk_{c.3} = "OK"$

$\beta_1 = 0.825$ $\beta_{1.ref} = "Equiv. Rect. Conc Stress Distrib., Tbl (22.2.2.4.3)"$

$c_{ewc} := \frac{a_{ewc}}{\beta_1}$ $c_{ewc} = 2.092 \cdot in$ $c_{h.e} := \frac{a_{h.e}}{\beta_1}$ $c_{h.e} = 1.901 \cdot in$ $c_{ref} = "Depth of N/A Axis, Eqn (22.2.2.4.1)"$

$c_{strip} := \frac{a_{strip}}{\beta_1}$ $c_{strip} = 2.947 \cdot in$

$\varepsilon_{ewc} := -0.003 + \frac{0.003}{c_{ewc}} \cdot d_{ewc}$ $\varepsilon_{ewc} = 0.0214$ $\varepsilon_{h.e} := -0.003 + \frac{0.003}{c_{h.e}} \cdot d_{h7.e}$ $\varepsilon_{h.e} = 0.0215$

$\varepsilon_{strip} := -0.003 + \frac{0.003}{c_{strip}} \cdot d_{strip}$ $\varepsilon_{strip} = 0.0209$ $\varepsilon_{ref} = "Tensile Strain, Extreme Layer"$

$\phi_{ewc} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{ewc} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{ewc} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.e} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{h.e} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.e} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi_{h.strip} := \min\left[\max\left[0.65, 0.65 + \left(\varepsilon_{strip} - 0.002\right) \cdot \left(\frac{250}{3}\right)\right], 0.9\right]$ $\phi_{h.strip} = 0.9$ $\phi_{ref} = "Strgth Reduc. Fact, Tbl (21.2.1)"$

$\phi Mn_{ewc} := \phi_{ewc} \cdot \min\left(A_{3.1}, A_{3.5}\right) \cdot f_y \cdot \left(d_{ewc} - \frac{a_{ewc}}{2}\right)$ $\phi Mn_{ewc} = 32 \cdot kip \cdot ft$

$\phi Mn_{h.e} := \phi_{h.e} \cdot \min\left(A_{3.2}, A_{3.5}\right) \cdot f_y \cdot \left(d_{h7.e} - \frac{a_{h.e}}{2}\right)$ $\phi Mn_{h.e} = 26.5 \cdot kip \cdot ft$

$\phi Mn_{strip} := \phi_{h.strip} \cdot A_{3.5} \cdot f_y \cdot \left(d_{strip} - \frac{a_{strip}}{2}\right)$ $\phi Mn_{strip} = 62.2 \cdot kip \cdot ft$ $\phi M_{p.ref} = "Flexural Strength, +ve, Sect (22.3)"$

$\rho_{t\_s.min} = 0.0018$ $\rho_{t\_s.min.ref} = "Temp/Shrink Min Reinf. Ratio, Tbl (24.4.3.2)"$

$\phi_V = 0.75$ $\phi_{V.ref} = "Shear Strgth Reduc. Fact, Tbl (21.2.1)"$

$\lambda = 0.85$ $\lambda_{ref} = "Mod. Factor for Sand LW Conc., Tbl (19.2.4.1(b))"$

$\lambda_{s.ewc} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{ewc}}{10 \cdot in}}}\right)$ $\lambda_{s.ewc} = 0.86$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.h.e} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{h7.e}}{10 \cdot in}}}\right)$ $\lambda_{s.h.e} = 0.89$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

$\lambda_{s.strip} := \min\left(1.0, \sqrt{\frac{2}{1 + \frac{d_{strip}}{10 \cdot in}}}\right)$ $\lambda_{s.strip} = 0.77$ $\lambda_{s.ref} = "Size Effect Fact, Eqn (22.5.5.1.3)"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
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Email: info@modularconnections.co

$Vc_{ewc.1} := 8\cdot\lambda\cdot\lambda_{s.ewc}\cdot\rho_{ewc}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{ewc}\right)$ $Vc_{ewc.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{ewc}\right)$ $Vc_{ewc.1} = 4.975\cdot kip$ $Vc_{ewc.2} = 19.387\cdot kip$

$\phi Vc_{ewc} := \min\left[\phi_V\cdot\min\left(Vc_{ewc.1}, Vc_{ewc.2}\right), 2\cdot\left(t_w\cdot d_{ewc}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{ewc} = 3.7\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{h.e.1} := 8\cdot\lambda\cdot\lambda_{s.h.e}\cdot\rho_{h.e}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{h7.e}\right)$ $Vc_{h.e.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{h7.e}\right)$ $Vc_{h.e.1} = 4.663\cdot kip$ $Vc_{h.e.2} = 17.676\cdot kip$

$\phi Vc_{h.e} := \min\left[\phi_V\cdot\min\left(Vc_{h.e.1}, Vc_{h.e.2}\right), 2\cdot\left(t_w\cdot d_{h7.e}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{h.e} = 3.5\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$Vc_{strip.1} := 8\cdot\lambda\cdot\lambda_{s.strip}^{\frac{1}{3}}\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{strip}\right)$ $Vc_{strip.2} := 5\cdot\lambda\cdot\left(\sqrt{f'_c\cdot psi}\cdot t_w\cdot d_{strip}\right)$ $Vc_{strip.1} = 39.347\cdot kip$ $Vc_{strip.2} = 26.799\cdot kip$

$\phi Vc_{h.strip} := \min\left[\phi_V\cdot\min\left(Vc_{strip.1}, Vc_{strip.2}\right), 2\cdot\left(t_w\cdot d_{h7.e}\right)\cdot\sqrt{f'_c\cdot psi}\right]$

$\phi Vc_{h.strip} = 8.3\cdot kip$ Geometry Max Shear Strength Eqn (22.5.1.2)

$$Uny_{L.ew} = \begin{array}{c} \begin{pmatrix} V_{h0} & V_{h1} & V_{h2} & M_{h0} & M_{h1} & M_{h2} \end{pmatrix} \\ \begin{pmatrix} 0.19 & 0.21 & 0.21 & 0.12 & 0.14 & 0.15 \\ 0.22 & 0.23 & 0.24 & 0.13 & 0.15 & 0.16 \\ 0.24 & 0.26 & 0.26 & 0.15 & 0.17 & 0.18 \\ 0.27 & 0.29 & 0.29 & 0.16 & 0.19 & 0.2 \\ 0.29 & 0.31 & 0.32 & 0.18 & 0.2 & 0.22 \\ 0.32 & 0.34 & 0.34 & 0.19 & 0.22 & 0.24 \\ 0.34 & 0.36 & 0.37 & 0.21 & 0.24 & 0.25 \\ 0.36 & 0.39 & 0.39 & 0.22 & 0.25 & 0.27 \\ 0.39 & 0.42 & 0.42 & 0.24 & 0.27 & 0.29 \end{pmatrix} \end{array} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Edge Column Strips

$chk_{c.4}$ = "OK"

$$Uny_{strip} = \begin{pmatrix} 0.1 & 0.1 & 0.11 & 0.07 & 0.07 & 0.08 \\ 0.12 & 0.12 & 0.12 & 0.08 & 0.08 & 0.09 \\ 0.13 & 0.13 & 0.13 & 0.08 & 0.09 & 0.1 \\ 0.11 & 0.1 & 0.09 & 0.09 & 0.1 & 0.11 \\ 0.12 & 0.16 & 0.16 & 0.1 & 0.11 & 0.12 \\ 0.13 & 0.17 & 0.17 & 0.11 & 0.12 & 0.13 \\ 0.14 & 0.18 & 0.18 & 0.12 & 0.13 & 0.14 \\ 0.15 & 0.2 & 0.2 & 0.12 & 0.14 & 0.15 \\ 0.16 & 0.21 & 0.21 & 0.13 & 0.15 & 0.16 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Interior Column Strip

$chk_{c.5}$ = "OK"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$Uny_{hdr} = \begin{pmatrix} 0.27 & 0.44 & 0.48 & 0.1 & 0.13 & 0.14 \\ 0.31 & 0.49 & 0.54 & 0.12 & 0.15 & 0.16 \\ 0.34 & 0.55 & 0.6 & 0.13 & 0.16 & 0.18 \\ 0.38 & 0.6 & 0.66 & 0.14 & 0.18 & 0.19 \\ 0.41 & 0.66 & 0.72 & 0.16 & 0.2 & 0.21 \\ 0.45 & 0.71 & 0.78 & 0.17 & 0.21 & 0.23 \\ 0.48 & 0.77 & 0.84 & 0.18 & 0.23 & 0.25 \\ 0.51 & 0.82 & 0.89 & 0.2 & 0.24 & 0.26 \\ 0.55 & 0.88 & 0.95 & 0.21 & 0.26 & 0.28 \end{pmatrix} \quad L = \begin{pmatrix} 12 \\ 14 \\ 16 \\ 18 \\ 20 \\ 22 \\ 24 \\ 26 \\ 28 \end{pmatrix} ft$$

Unity for Header

$chk_{c.6}$ = "OK"

$Vuh0^T = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5)\cdot kip$

$Vuh1^T = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5)\cdot kip$

$Vuh2^T = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5)\cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Wall Plates: $No_{plates} := 1 \quad V_{u.Wall.PLh0} := 2\dfrac{Vuh0\cdot H_{wall_0}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh0} := V_{u.Wall.PLh0}$

$V_{u.Floor.PLh0}{}^T = (3.4 \quad 3.9 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6 \quad 6.4 \quad 6.9)\cdot kip$

Wall Plates: $V_{u.Wall.PLh1} := 2\dfrac{Vuh1\cdot H_{wall_1}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh1} := V_{u.Wall.PLh1}$

$V_{u.Floor.PLh1}{}^T = (3.8 \quad 4.3 \quad 4.8 \quad 5.3 \quad 5.7 \quad 6.2 \quad 6.7 \quad 7.2 \quad 7.6)\cdot kip$

Wall Plates: $V_{u.Wall.PLh2} := 2\dfrac{Vuh2\cdot H_{wall_2}}{W_{ew}\cdot No_{plates}} \quad V_{u.Floor.PLh2} := V_{u.Wall.PLh2}$

$V_{u.Floor.PLh2}{}^T = (4.1 \quad 4.6 \quad 5.1 \quad 5.6 \quad 6.1 \quad 6.6 \quad 7.1 \quad 7.6 \quad 8.1)\cdot kip$

$A_{cv.b.3b.1}{}^T = (592 \quad 688 \quad 784 \quad 880 \quad 976 \quad 1072 \quad 1168 \quad 1264 \quad 1360)\cdot in^2$

$\rho_{b.2}{}^T = (0.00115 \quad 0.00099 \quad 0.00087 \quad 0.00077 \quad 0.0007 \quad 0.00063 \quad 0.00058 \quad 0.00054 \quad 0.0005)$

Check End Wall with ACI Special Seismic Considerations

Section 21.6.2.2 two curtains of reinforcement are required if in plane shear (factored) assigned to the wall exceeds:

$2\cdot A_{cv.b.3b.1}\cdot\sqrt{f'_c\cdot psi} \quad \Longrightarrow \quad 2\cdot t_w\cdot\sqrt{f'_c\cdot psi} = 6.4\cdot\dfrac{kip}{ft}$

Seismic Shear Force $v_{u.5d.1.2h0} := \dfrac{V_{u.Wall.PLh0}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.1.2h0}{}^T = (2.2 \quad 2.5 \quad 2.8 \quad 3.1 \quad 3.3 \quad 3.6 \quad 3.9 \quad 4.2 \quad 4.5)\cdot\dfrac{kip}{ft}$

$v_{u.5d.2.2h1} := \dfrac{V_{u.Wall.PLh1}}{\left\lceil\left(L_{ew.mn}\right) - 3.5in\right\rceil}$

$v_{u.5d.2.2h1}{}^T = (2.5 \quad 2.8 \quad 3.1 \quad 3.4 \quad 3.7 \quad 4 \quad 4.3 \quad 4.6 \quad 5)\cdot\dfrac{kip}{ft}$

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Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$v_{u.5d.3.2h2} := \frac{V_{u.Wall.PLh2}}{\left\lceil \left(L_{ew.mn}\right) - 3.5in \right\rceil}$$

$$v_{u.5d.3.2h2}^{T} = (2.6 \;\; 3 \;\; 3.3 \;\; 3.6 \;\; 4 \;\; 4.3 \;\; 4.6 \;\; 5 \;\; 5.3)\cdot\frac{kip}{ft}$$

$$Unity_{5d1.2.3} := stack\left(\frac{v_{u.5d.1.2h0}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.2.2h1}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}, \frac{v_{u.5d.3.2h2}^{T}}{2\cdot t_w\cdot\sqrt{f'_c\cdot psi}}\right)^{T} \qquad Hwall := \left(H_{wall_0} \;\; H_{wall_1} \;\; H_{wall_2}\right)$$

$$Unity_{5d1.2.3}^{T} = \begin{pmatrix} 34.71 & 39.02 & 43.34 & 47.66 & 51.97 & 56.29 & 60.61 & 64.93 & 69.24 \\ 38.56 & 43.36 & 48.16 & 52.95 & 57.75 & 62.55 & 67.34 & 72.14 & 76.94 \\ 41.13 & 46.25 & 51.37 & 56.48 & 61.6 & 66.72 & 71.83 & 76.95 & 82.07 \end{pmatrix}\cdot\% \qquad Hwall^{T} = \begin{pmatrix} 9 \\ 10 \\ 10.67 \end{pmatrix} ft$$

$$L^{T} = (12.00 \;\; 14.00 \;\; 16.00 \;\; 18.00 \;\; 20.00 \;\; 22.00 \;\; 24.00 \;\; 26.00 \;\; 28.00)\,ft$$

$chk_{5d.1} := if\left(1.0 \geq \max\left(Unity_{5d1.2.3}\right), "OK", "NG"\right)$ $chk_{5d.1} = "OK"$

Check wall for combined wind. (2 way plate for out - of - plane loads)

$w_{plate} := 16in$ Distance from the end of the Side Wall to the 1st Wall To Floor Connection Plate

$t_w = 4\cdot in$ Wall Thickness $w_{plate} = 16\cdot in$ Plate Width

$$b_{eff.5d} := \begin{pmatrix} 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \\ 4\cdot t_w + w_{plate} \end{pmatrix} \qquad b_{eff.5d} = \begin{pmatrix} 32 \\ 32 \\ 32 \end{pmatrix}\cdot in$$

$b_{eff.5d.4} := b_{eff.5d_0}$ $b_{eff.5d.5} := b_{eff.5d_1}$ $b_{eff.5d.6} := b_{eff.5d_2}$ Effective Plate Width

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas

$a_{5d4} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.4} := \left(H_{wall_0} - t_{floor}\right)$	$b_{5d.4} = 8.3\,ft$	For Wall Panel Height of	
$a\_b_{5d.4} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.4} := H_{wall_0} - t_{floor}$	$l_{c.5d.4} = 8.3\,ft$	$H_{wall_0} = 9\,ft$	
$a_{5d5} := W_{ew}$	$a_{5d5} = 11.7\,ft$	$b_{5d.5} := \left(H_{wall_1} - t_{floor}\right)$	$b_{5d.5} = 9.3\,ft$	For Wall Panel Height of	
$a\_b_{5d.5} := \frac{a_{5d4}}{b_{5d.4}}$	$a\_b_{5d.4} = 1.4$	$l_{c.5d.5} := H_{wall_1} - t_{floor}$	$l_{c.5d.5} = 9.3\,ft$	$H_{wall_1} = 10\,ft$	
$a_{5d6} := W_{ew}$	$a_{5d4} = 11.7\,ft$	$b_{5d.6} := \left(H_{wall_2} - t_{floor}\right)$	$b_{5d.6} = 10\,ft$	For Wall Panel Height of	
$a\_b_{5d.6} := \frac{a_{5d6}}{b_{5d.6}}$	$a\_b_{5d.6} = 1.2$	$l_{c.5d.6} := H_{wall_2} - t_{floor}$	$l_{c.5d.6} = 10\,ft$	$H_{wall_2} = 10.7ft$	

$$l_{c.5d} := \begin{pmatrix} l_{c.5d.4} \\ l_{c.5d.5} \\ l_{c.5d.6} \end{pmatrix} \qquad l_{c.5d} = \begin{pmatrix} 8.33 \\ 9.33 \\ 10 \end{pmatrix} ft$$

$P_{5.5} := \max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)$ $P_{5.5} = 64.5\cdot psf$

$\beta_{5d.4} = 0.5$ $\beta_{5d.5} = 0.5$ $\beta_{5d.6} = 0.4$ $k_{5.5} := 0.8$ conservatively, unrestrained from rotation at both ends ACI 318-19 11.5.3.2

$\beta_{5d.4} = 0.5$ $\sigma_{5d.4} := \frac{\beta_{5d.4}\cdot P_{5.5}\cdot\left(l_{c.5d.4}\right)^2}{\left(t_w\right)^2} = 18.25\cdot ksf$ $S_{5d.4} := \frac{\left(t_w\right)^2}{6}\cdot 1ft$ $S_{5d.4} = 32\cdot in^3$

$Mu_{5d.4} := \sigma_{5d.4}\cdot S_{5d.4}$ $Mu_{5d.4} = 4.1\cdot kip\cdot in$

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Assume only (1) end wall for resistance to lateral load

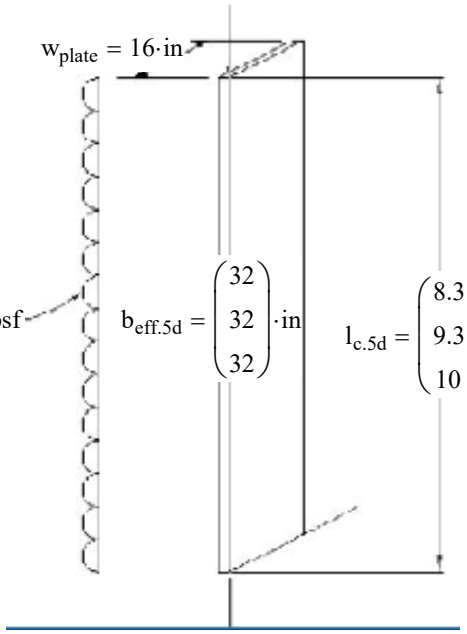
$P_{5d.4} := 2 \cdot V_{u.Wall.PLh0}$

$P_{5d.4}^{T} = (6.9 \;\; 7.7 \;\; 8.6 \;\; 9.5 \;\; 10.3 \;\; 11.2 \;\; 12 \;\; 1$ $P_{5d.4max} := \max(2 \cdot V_{u.Wall.PLh0})$ $P_{5d.4max} = 13.7 \cdot kip$

$A_{g.5d.4} := b_{eff.5d.4} \cdot t_w$ $A_{g.5d.4} = 128 \cdot in^2$ $b_{eff.5d.4} = 32 \cdot in$

$V_{u.Floor.PLh0}^{T} = (3.4 \;\; 3.9 \;\; 4.3 \;\; 4.7 \;\; 5.2 \;\; 5.6 \;\; 6 \;\; 6.4 \;\; 6.9) \cdot kip$

$\beta_{5d.5} = 0.5$ $\sigma_{5d.5} := \dfrac{\beta_{5d.5} \cdot P_{5.5} \cdot (l_{c.5d.5})^2}{(t_w)^2} = 22.89 \cdot ksf$ $\beta_{5d.6} = 0.4$ $\sigma_{5d.6} := \dfrac{\beta_{5d.6} \cdot P_{5.5} \cdot (l_{c.5d.6})^2}{(t_w)^2} = 20.96 \cdot ksf$



$S_{5d.5} := \dfrac{(t_w)^2}{6} \cdot 1 ft$ $S_{5d.5} = 32 \cdot in^3$

$Mu_{5d.5} := \sigma_{5d.5} \cdot S_{5d.5}$ $Mu_{5d.5} = 5.1 \cdot kip \cdot in$

Assume only (1) end wall for resistance to lateral load

$P_{5d.5} := 2 \cdot V_{u.Wall.PLh1}$

$P_{5d.5max} := \max(2 \cdot V_{u.Wall.PLh1})$ $P_{5d.5max} = 15.3 \cdot kip$

$A_{g.5d.5} := b_{eff.5d.5} \cdot t_w$ $A_{g.5d.5} = 128 \cdot in^2$

$b_{eff.5d.5} = 32 \cdot in$

$S_{5d.6} := \dfrac{(t_w)^2}{6} \cdot 1 ft$ $S_{5d.6} = 32 \cdot in^3$

$Mu_{5d.6} := \sigma_{5d.6} \cdot S_{5d.6}$ $Mu_{5d.6} = 4.7 \cdot kip \cdot in$

$P_{5d.5}^{T} = (7.7 \;\; 8.6 \;\; 9.6 \;\; 10.5 \;\; 11.5 \;\; 12.4 \;\; 13.4 \;\; 14.3 \;\; 15.3) \cdot kip$

$V_{u.Floor.PLh1}^{T} = (3.8 \;\; 4.3 \;\; 4.8 \;\; 5.3 \;\; 5.7 \;\; 6.2 \;\; 6.7 \;\; 7.2 \;\; 7.6) \cdot kip$

Assume only (1) end wall for resistance to lateral load

$P_{5d.6} := 2 \cdot V_{u.Wall.PLh2}$ $P_{5d.6}^{T} = (8.2 \;\; 9.2 \;\; 10.2 \;\; 11.2 \;\; 12.2 \;\; 13.2 \;\; 14.3 \;\; 15.3 \;\; 16.3) \cdot kip$

$P_{5d.6max} := \max(2 \cdot V_{u.Wall.PLh2})$ $P_{5d.6max} = 16.3 \cdot kip$

$A_{g.5d.6} := b_{eff.5d.6} \cdot t_w$ $A_{g.5d.6} = 128 \cdot in^2$ $b_{eff.5d.6} = 32 \cdot in$

$V_{u.Floor.PLh2}^{T} = (4.1 \;\; 4.6 \;\; 5.1 \;\; 5.6 \;\; 6.1 \;\; 6.6 \;\; 7.1 \;\; 7.6 \;\; 8.1) \cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$\phi_c := 0.65$ Compression Controled Axial Strength

$P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} = 229.2 \cdot kip \cdot in$ $Mu_{5d.4} = 4.1 \cdot kip \cdot in$

$chk_{5d.2} := if\left(P_{5d.4max} \cdot \dfrac{l_{c.5d.4}}{6} \geq Mu_{5d.4}, "OK", "NG"\right)$ $chk_{5d.2} = "OK"$

$\phi P_{5d.4} := 0.45 \cdot \phi_c \cdot f'_c \cdot A_{g.5d.4} \cdot \left[1 - \left(\dfrac{k_{5.5} \cdot l_{c.5d.4}}{32 \cdot t_w}\right)^2\right]$ $\phi P_{5d.4} = 102.7 \cdot kip$

ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

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$Unity_{5d.2} := \frac{P_{5d.4}}{\phi P_{5d.4}}$ $Unity_{5d.2}^{T} = (6.7 \quad 7.5 \quad 8.4 \quad 9.2 \quad 10.1 \quad 10.9 \quad 11.7 \quad 12.6 \quad 13.4)\cdot\%$

$chk_{5d.3} := if(1.0 \geq 2\,max(Unity_{5d.2}), "OK", "NG")$ $chk_{5d.3} = "OK"$

$P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} = 285.2\cdot kip\cdot in$ $Mu_{5d.5} = 5.1\cdot kip\cdot in$

$chk_{5d.4} := if\left(P_{5d.5max}\cdot\frac{l_{c.5d.5}}{6} \geq Mu_{5d.5}, "OK", "NG"\right)$ $chk_{5d.4} = "OK"$

$\phi P_{5d.5} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.5}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.5}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.5} = 85.9\cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.3} := \frac{P_{5d.5}}{\phi P_{5d.5}}$ $Unity_{5d.3}^{T} = (8.9 \quad 10 \quad 11.1 \quad 12.2 \quad 13.3 \quad 14.5 \quad 15.6 \quad 16.7 \quad 17.8)\cdot\%$

$chk_{5d.5} := if(1.0 \geq 2\,max(Unity_{5d.3}), "OK", "NG")$ $chk_{5d.5} = "OK"$

$P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} = 325.9\cdot kip\cdot in$ $Mu_{5d.6} = 4.7\cdot kip\cdot in$

$chk_{5d.6} := if\left(P_{5d.6max}\cdot\frac{l_{c.5d.6}}{6} \geq Mu_{5d.6}, "OK", "NG"\right)$ $chk_{5d.6} = "OK"$

$\phi P_{5d.6} := 0.45\cdot\phi_c\cdot f'_c\cdot A_{g.5d.6}\cdot\left[1 - \left(\frac{k_{5.5}\cdot l_{c.5d.6}}{32\cdot t_w}\right)^2\right]$ $\phi P_{5d.6} = 73.7\cdot kip$ ACI 318-19 eqn (14.5.4.2, 11.5.3.1)

$Unity_{5d.4} := \frac{P_{5d.6}}{\phi P_{5d.6}}$ $Unity_{5d.4}^{T} = (11.1 \quad 12.5 \quad 13.8 \quad 15.2 \quad 16.6 \quad 18 \quad 19.4 \quad 20.7 \quad 22.1)\cdot\%$

$chk_{5d.7} := if(1.0 \geq 2\,max(Unity_{5d.4}), "OK", "NG")$ $chk_{5d.7} = "OK"$

Ref. "Design of Concrete Structure" by Winter & Nelson

$\rho_{min.5e.1} := 0.0020$ for WWF smaller than W31 or D31 ACI 318-19 11.6.1 $Amin_{5e.1} := t_w\cdot\rho_{min.5e.1}$ $Amin_{5e.1} = 0.096\cdot\frac{in^2}{ft}$

$\alpha_{c.5e.1h0}^{T} = (1.8 \quad 1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_0} = 9\,ft$ $Vn_{c.5e.1h0} = \alpha_{c.5e.1h0}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h0}^{T} = (2.6 \quad 2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3)\cdot kip$

$V_{L.ewh0}^{T} = (0.7 \quad 0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.4)\cdot kip$

$Unity_{5e.1.h0}^{T} = (28 \quad 31.9 \quad 36 \quad 40.2 \quad 44.6 \quad 49.1 \quad 53.7 \quad 58.5 \quad 63.4)\cdot\%$

$chk_{5e.1} := if(1.0 \geq max(Unity_{5e.1.h0}), "OK", "NG")$ $chk_{5e.1} = "OK"$

$\alpha_{c.5e.1h1}^{T} = (1.8 \quad 1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_1} = 10\,ft$ $Vn_{c.5e.1h1} = \alpha_{c.5e.1h1}\cdot\left[0.5\cdot\phi_V\cdot\lambda\cdot\sqrt{f'_c\cdot psi}\cdot(t_w\cdot d_{ewc})\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h1}^{T} = (2.6 \quad 2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.3 \quad 2.2)\cdot kip$

$V_{L.ewh1}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6)\cdot kip$

$Unity_{5e.1.h1}^{T} = (30.4 \quad 34.8 \quad 39.3 \quad 44 \quad 48.9 \quad 53.9 \quad 59.1 \quad 64.5 \quad 70.2)\cdot\%$

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$chk_{5e.2} := if\left(1.0 \geq max\left(Unity_{5e.1.h1}\right), "OK", "NG"\right)$ $chk_{5e.2} = "OK"$

$\alpha_{c.5e.1h2}^{T} = (1.7 \quad 1.7 \quad 1.7 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.6 \quad 1.5 \quad 1.5)$

$L^{T} = (12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26 \quad 28)\,ft$

$H_{wall_2} = 10.7\,ft \qquad Vn_{c.5e.1h2} = \alpha_{c.5e.1h2} \cdot \left[0.5 \cdot \phi_V \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot \left(t_w \cdot d_{ewc}\right)\right]$ ACI 318-19 11.5.4.3

$Vn_{c.5e.1h2}^{T} = (2.5 \quad 2.5 \quad 2.4 \quad 2.4 \quad 2.4 \quad 2.3 \quad 2.3 \quad 2.2 \quad 2.2) \cdot kip$

$V_{L.ewh2}^{T} = (0.8 \quad 0.9 \quad 1 \quad 1.1 \quad 1.2 \quad 1.3 \quad 1.4 \quad 1.5 \quad 1.6) \cdot kip$

$Unity_{5e.1.h2}^{T} = (31 \quad 35.6 \quad 40.2 \quad 45.1 \quad 50.1 \quad 55.4 \quad 60.9 \quad 66.5 \quad 72.5) \cdot \%$

$chk_{5e.3} := if\left(1.0 \geq max\left(Unity_{5e.1.h2}\right), "OK", "NG"\right)$ $chk_{5e.3} = "OK"$

$P_{5d.4}^{T} = (6.9 \quad 7.7 \quad 8.6 \quad 9.5 \quad 10.3 \quad 11.2 \quad 12 \quad 12.9 \quad 13.7) \cdot kip$

$Mu_{5d.4} = 4.06 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.4} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.4}} \qquad \rho_{5e.4} = 0.00094$

$u_{5e.4} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.4} = 15.7 \qquad d_{5e.4} := \frac{t_w}{2} \qquad d_{5e.4} = 2 \cdot in \qquad e'_{5e.4} := \frac{Mu_{5d.4}}{P_{5d.4}}$

$e'_{5e.4}{}^{T} = (0.59 \quad 0.52 \quad 0.47 \quad 0.43 \quad 0.39 \quad 0.36 \quad 0.34 \quad 0.31 \quad 0.29) \cdot in$

$$P_{d.5e.4} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \cdot \left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]$$

$x_{5e.4} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.4} \cdot d_{5e.4} \qquad x_{5e.4} = 159.1 \cdot kip$

$$P_{d.5e.4} := x_{5e.4} \cdot \left[\left[-\rho_{5e.4} \cdot u_{5e.4} + 1 - \frac{e'_{5e.4}}{d_{5e.4}} + \sqrt{\left(1 - \frac{e'_{5e.4}}{d_{5e.4}}\right)^2 + 2 \cdot \frac{e'_{5e.4}}{d_{5e.4}} \cdot \rho_{5e.4} \cdot u_{5e.4}}\right]\right]$$

$P_{d.5e.4}^{T} = (223.2 \quad 233.4 \quad 241.6 \quad 248.3 \quad 253.9 \quad 258.7 \quad 262.8 \quad 266.3 \quad 269.4) \cdot kip$

$Unity_{5e.4} := \frac{P_{5d.4}}{P_{d.5e.4}} \qquad Unity_{5e.4}^{T} = (3.1 \quad 3.3 \quad 3.6 \quad 3.8 \quad 4.1 \quad 4.3 \quad 4.6 \quad 4.8 \quad 5.1) \cdot \%$

$chk_{5e.4} := if\left(1.0 \geq max\left(Unity_{5e.4}\right), "OK", "NG"\right)$ $chk_{5e.4} = "OK"$

$P_{5d.5}^{T} = (7.7 \quad 8.6 \quad 9.6 \quad 10.5 \quad 11.5 \quad 12.4 \quad 13.4 \quad 14.3 \quad 15.3) \cdot kip$

$Mu_{5d.5} = 5.09 \cdot kip \cdot in \qquad f'_c = 4500 \cdot psi \qquad \phi_c = 0.65 \qquad \rho_{5e.5} := \frac{A_{s.4a} \cdot ft}{t_w \cdot b_{eff.5d.5}} \qquad \rho_{5e.5} = 0.00094$

$u_{5e.5} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5e.5} = 15.7 \qquad d_{5e.5} := \frac{t_w}{2} \qquad d_{5e.5} = 2 \cdot in \qquad e'_{5e.5} := \frac{Mu_{5d.5}}{P_{5d.5}}$

$e'_{5e.5}{}^{T} = (0.66 \quad 0.59 \quad 0.53 \quad 0.48 \quad 0.44 \quad 0.41 \quad 0.38 \quad 0.36 \quad 0.33) \cdot in$

$$P_{d.5e.5} = \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \cdot \left[-\rho_{5e.5} \cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2 \cdot \frac{e'_{5e.5}}{d_{5e.5}} \cdot \rho_{5e.5} \cdot u_{5e.5}}\right]$$

$x_{5e.5} := \phi_c \cdot 0.85 \cdot f'_c \cdot b_{eff.5d.5} \cdot d_{5e.5} \qquad x_{5e.5} = 159.1 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF/200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$P_{d.5e.5} := x_{5e.5}\cdot\left[\left[-\rho_{5e.5}\cdot u_{5e.5} + 1 - \frac{e'_{5e.5}}{d_{5e.5}} + \sqrt{\left(1 - \frac{e'_{5e.5}}{d_{5e.5}}\right)^2 + 2\cdot\frac{e'_{5e.5}}{d_{5e.5}}\cdot\rho_{5e.5}\cdot u_{5e.5}}\right]\right]$$

$P_{d.5e.5}^{T} = (211.3 \quad 222.9 \quad 232.1 \quad 239.7 \quad 246 \quad 251.3 \quad 255.9 \quad 259.9 \quad 263.4)\cdot kip$

$Unity_{5e.5} := \frac{P_{5d.5}}{P_{d.5e.5}}$ $\quad Unity_{5e.5}^{T} = (3.6 \quad 3.9 \quad 4.1 \quad 4.4 \quad 4.7 \quad 4.9 \quad 5.2 \quad 5.5 \quad 5.8)\cdot\%$

$chk_{5e.5} := if\left(1.0 \geq \max\left(Unity_{5e.5}\right), "OK", "NG"\right)$ $\quad chk_{5e.5} = "OK"$

$P_{5d.6}^{T} = (8.2 \quad 9.2 \quad 10.2 \quad 11.2 \quad 12.2 \quad 13.2 \quad 14.3 \quad 15.3 \quad 16.3)\cdot kip$

$Mu_{5d.6} = 4.66\cdot kip\cdot in$ $\quad f'_c = 4500\cdot psi$ $\quad \phi_c = 0.65$ $\quad \rho_{5e.6} := \frac{A_{s.4a}\cdot ft}{t_w\cdot b_{eff.5d.6}}$ $\quad \rho_{5e.6} = 0.00094$

$u_{5e.6} := \frac{f_y}{0.85\cdot f'_c}$ $\quad u_{5e.6} = 15.7$ $\quad d_{5e.6} := \frac{t_w}{2}$ $\quad d_{5e.6} = 2\cdot in$ $\quad e'_{5e.6} := \frac{Mu_{5d.6}}{P_{5d.6}}$

$e'^{T}_{5e.6} = (0.57 \quad 0.51 \quad 0.46 \quad 0.42 \quad 0.38 \quad 0.35 \quad 0.33 \quad 0.3 \quad 0.29)\cdot in$

$$P_{d.5e.6} = \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}\cdot\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]$$

$x_{5e.6} := \phi_c\cdot 0.85\cdot f'_c\cdot b_{eff.5d.6}\cdot d_{5e.6}$ $\quad x_{5e.6} = 159.1\cdot kip$

$$P_{d.5e.6} := x_{5e.6}\cdot\left[\left[-\rho_{5e.6}\cdot u_{5e.6} + 1 - \frac{e'_{5e.6}}{d_{5e.6}} + \sqrt{\left(1 - \frac{e'_{5e.6}}{d_{5e.6}}\right)^2 + 2\cdot\frac{e'_{5e.6}}{d_{5e.6}}\cdot\rho_{5e.6}\cdot u_{5e.6}}\right]\right]$$

$P_{d.5e.6}^{T} = (226.1 \quad 236 \quad 243.9 \quad 250.4 \quad 255.8 \quad 260.4 \quad 264.4 \quad 267.8 \quad 270.8)\cdot kip$

$Unity_{5e.6} := \frac{P_{5d.6}}{P_{d.5e.6}}$ $\quad Unity_{5e.6}^{T} = (3.6 \quad 3.9 \quad 4.2 \quad 4.5 \quad 4.8 \quad 5.1 \quad 5.4 \quad 5.7 \quad 6)\cdot\%$

$chk_{5e.6} := if\left(1.0 \geq \max\left(Unity_{5e.6}\right), "OK", "NG"\right)$ $\quad chk_{5e.6} = "OK"$

Check side wall for it's contribution

$Embed_{5f.1} := V_{u.Floor.PLh0}^{T} = (3.4 \quad 3.9 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6 \quad 6.4 \quad 6.9)\cdot kip$

$w_{DL.5f.1.mx} := DL_{r.mx}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx} = \begin{pmatrix} 676.3 \\ 715.6 \\ 741.8 \end{pmatrix}\cdot plf$

$w_{u.5f.1.DL.mx} := LF_{DL.min}\cdot w_{DL.5f.1.mx}$ $\quad w_{u.5f.1.DL.mx} = \begin{pmatrix} 608.6 \\ 644 \\ 667.6 \end{pmatrix}\cdot plf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$w_{DL.5f.1.mn} := DL_{r.mn}\cdot\frac{W_r}{2} + \gamma_{LW}\cdot t_w\cdot H_{wall}$ $\quad w_{DL.5f.1.mx}^{T} = (676.3 \quad 715.6 \quad 741.8)\cdot plf$

$w_{u.5f.1.DL.mn} := LF_{DL.min}\cdot w_{DL.5f.1.mn}$ $\quad w_{u.5f.1.DL.mx}^{T} = (608.6 \quad 644 \quad 667.6)\cdot plf$

Compression Side: $R_y := w_{DL.5f.1.mn} = \begin{pmatrix} 634.3 \\ 673.6 \\ 699.8 \end{pmatrix}\cdot plf$ <==> Assume its distributed for width = height/3 $w_{5f.1} := \frac{\left(H_{wall} - t_{floor}\right)}{3}$

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$w_{5f.1} = \begin{pmatrix} 2.8 \\ 3.1 \\ 3.3 \end{pmatrix} ft$ Therefore: minimum of $No_{plates} = 1$ embed plates ==> $R_{embed} := \frac{\begin{pmatrix} w_{DL.5f.1.mx} \\ -w_{DL.5f.1.mx} \end{pmatrix}}{No_{plates}} = \begin{bmatrix} \begin{pmatrix} 676.25 \\ 715.58 \\ 741.81 \end{pmatrix} \\ \begin{pmatrix} -676.25 \\ -715.58 \\ -741.81 \end{pmatrix} \end{bmatrix} \cdot plf$

$w_{up} := \frac{\left|P_{wind_3}\right| \cdot W_{ew}}{2}$ $w_{up} = 377.5 \cdot plf$ $w_{u.up} := w_{up}$ $w_{u.up} = 377.5 \cdot plf$

Compression Side

$P_{5f.2.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h0} = (5.2 \quad 5.6 \quad 6 \quad 6.5 \quad 6.9 \quad 7.3 \quad 7.8 \quad 8.2 \quad 8.6) \cdot kip$

$P_{5f.2.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h1} = (5.4 \quad 5.8 \quad 6.3 \quad 6.7 \quad 7.1 \quad 7.6 \quad 8 \quad 8.4 \quad 8.8) \cdot kip$

$P_{5f.2.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} + Embed_{5f.1}$

$P_{5f.2.h2} = (5.6 \quad 6 \quad 6.4 \quad 6.9 \quad 7.3 \quad 7.7 \quad 8.1 \quad 8.6 \quad 9) \cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF/ 200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Tension Side

$P_{5f.3.h0} := w_{DL.5f.1.mx_0} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h0} = (-1.7 \quad -2.1 \quad -2.6 \quad -3 \quad -3.4 \quad -3.8 \quad -4.3 \quad -4.7 \quad -5.1) \cdot kip$

$P_{5f.3.h1} := w_{DL.5f.1.mx_1} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h1} = (-1.5 \quad -1.9 \quad -2.3 \quad -2.8 \quad -3.2 \quad -3.6 \quad -4 \quad -4.5 \quad -4.9) \cdot kip$

$P_{5f.3.h2} := w_{DL.5f.1.mx_2} \cdot \frac{W_{ew}}{2} - w_{u.up} \cdot \frac{W_{ew}}{2} - Embed_{5f.1}$

$P_{5f.3.h2} = (-1.3 \quad -1.7 \quad -2.2 \quad -2.6 \quad -3 \quad -3.5 \quad -3.9 \quad -4.3 \quad -4.7) \cdot kip$

$W_{u.5f.1} := \left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| = 64.5 \cdot psf$ $Mu_{5f.1} := \frac{W_{u.5f.1} \cdot (H_E - 8in)^2}{8} = \begin{pmatrix} 7.5 \\ 9.3 \\ 10.6 \end{pmatrix} \cdot \frac{kip \cdot in}{ft}$

$d_{5.4} := 0.5 \cdot t_w$ $d_{5.4} = 2 \cdot in$ $x_{5f.2} := 0.00349$ Initial Guess for reinforcement ratio

$f(x_{5f.2}) := \frac{\frac{\max(Mu_{5f.1})}{\left(\frac{kip \cdot ft}{ft}\right)}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d_{5.4}}{in}\right)^2 \cdot (1 - 8.85 \cdot x_{5f.2})} - x_{5f.2}$ $\rho_{5f.2} := \frac{root(f(x_{5f.2}), x_{5f.2})}{ft}$ $\rho_{5f.2} = 0.00426 \cdot \frac{1}{ft}$

Reinforcement Ratio Required for out of plane Wind

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$A_{s.req1.5f.2.} := \rho_{5f.2} \cdot 12in \cdot d_{5.4}$ $A_{s.req1.5f.2.} = 0.102 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for out of plane Wind

$$A_{s.req2.5f.2} := \max\left(\frac{\max\left(\max\left(P_{5f.3.h0}, \left|\min\left(P_{5f.3.h0}\right)\right|\right), \max\left(P_{5f.3.h1}, \left|\min\left(P_{5f.3.h1}\right)\right|\right), \max\left(P_{5f.3.h2}, \left|\min\left(P_{5f.3.h2}\right)\right|\right)\right)}{0.9 \cdot f_y \cdot \frac{W_{ew}}{2}}\right)$$

$A_{s.req2.5f.2} = 0.01629 \cdot \frac{in^2}{ft}$ Area of Reinforcement Required for Tension Force from Uplift

$A_{req.5f.2} := A_{s.req1.5f.2.} + A_{s.req2.5f.2}$ $A_{req.5f.2} = 0.118 \cdot \frac{in^2}{ft}$ Total Area of Reinforcement Required

$A_{s.4a} = 0.12 \cdot \frac{in^2}{ft}$ Area of Steel Provided

$chk_{5.8} := if\left(A_{req.5f.2} \le A_{s.4a}, "OK", "NG"\right)$ $chk_{5.8} = "OK"$

Check P-Δ Effects

$E_{c.spec} = 2991 \cdot ksi$ $I_{g.5g.1} := \frac{1}{12} \cdot 12in \cdot \left(t_w\right)^3$ $I_{g.5g.1} = 64 \cdot in^4$

$P_{cr.5g.1} := \frac{\pi^2 \cdot E_{c.spec} \cdot I_{g.5g.1}}{H_E^2}$ $P_{cr.5g.1} = \begin{pmatrix} 146 \\ 119.5 \\ 105.6 \end{pmatrix} \cdot kip$ $Ma_{g.2} := \frac{\left|\max\left(\left|P_{W.c}\right|, \left|P_{L.c}\right|\right)\right| \cdot \left(H_{wall}\right)^2}{8} \cdot 12in$ $Ma_{g.2} = \begin{pmatrix} 652.6 \\ 805.7 \\ 916.7 \end{pmatrix} \cdot lb \cdot ft$

$Mb_{5g.2} := Ma_{g.2}$ $Mb_{5g.2} = \begin{pmatrix} 0.7 \\ 0.8 \\ 0.9 \end{pmatrix} \cdot kip \cdot ft$ $C_{m.5g.2} := 0.6 + 0.4 \cdot \frac{Ma_{g.2}}{Mb_{5g.2}}$ $C_{m.5g.2} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$

$$\delta_{ns.5.6} := \frac{C_{m.5g.2}}{1 - \frac{\max\left(\max\left(P_{5f.2.h0}, \left|\min\left(P_{5f.2.h0}\right)\right|\right), \max\left(P_{5f.2.h1}, \left|\min\left(P_{5f.2.h1}\right)\right|\right), \max\left(P_{5f.2.h2}, \left|\min\left(P_{5f.2.h2}\right)\right|\right)\right)}{0.5 \cdot P_{cr.5g.1}}}$$

$\delta_{ns.5.6} = \begin{pmatrix} 1.1 \\ 1.2 \\ 1.2 \end{pmatrix}$

$Ma_{g.3} := \begin{pmatrix} \delta_{ns.5.6_0} \cdot Ma_{g.2_0} \\ \delta_{ns.5.6_1} \cdot Ma_{g.2_1} \\ \delta_{ns.5.6_2} \cdot Ma_{g.2_2} \end{pmatrix}$ $Ma_{g.3} = \begin{pmatrix} 744.4 \\ 948.6 \\ 1105 \end{pmatrix} \cdot lb \cdot ft$ $f_r = 427.6 \cdot psi$ $M_{cr.5g.2} := \frac{f_r \cdot I_{g.5g.1}}{d_{5.4}}$ $M_{cr.5g.2} = 1140.4\ ft \cdot \frac{lb \cdot ft}{ft}$

$chk_{5.9} := if\left(M_{cr.5g.2} \le \max\left(Ma_{g.3}\right), "Cracking Occurs", "Cracking Doesn't Occur"\right)$ $chk_{5.9} = "Cracking Doesn't Occur"$

$\Delta_{5g.1} := \frac{Ma_{g.3}}{M_{cr.5g.2}} \cdot \frac{5 \cdot \left(\delta_{ns.5.6} \cdot M_{cr.5g.2}\right) \cdot \left(\left(H_{wall} - t_{floor}\right)\right)^2}{48 \cdot E_{c.spec} \cdot I_{g.5g.1}}$ $\Delta_{5g.1} = \begin{pmatrix} 0.21 \\ 0.27 \\ 0.31 \end{pmatrix} \cdot in$ $Mu_{5.5} := Mu_{5f.1} \cdot ft$ $Mu_{5.5} = \begin{pmatrix} 7.5 \\ 9.3 \\ 10.6 \end{pmatrix} \cdot kip \cdot in$

$Mu_{5.6} := \max\left(Mu_{5.5}\right) + Embed_{5f.1} \cdot \left(\max\left(\Delta_{5g.1}\right) \cdot \max\left(\delta_{ns.5.6}\right)\right)$

$Mu_{5.6} = (11.9 \quad 12.1 \quad 12.2 \quad 12.4 \quad 12.6 \quad 12.7 \quad 12.9 \quad 13.1 \quad 13.2) \cdot kip \cdot in$

$P_{5.8} := Embed_{5f.1}$

$P_{5.8} = (3.4 \quad 3.9 \quad 4.3 \quad 4.7 \quad 5.2 \quad 5.6 \quad 6 \quad 6.4 \quad 6.9) \cdot kip$

$e`_{5.6} := \frac{Mu_{5.6}}{P_{5.8}}$ $e`_{5.6} = (3.5 \quad 3.1 \quad 2.8 \quad 2.6 \quad 2.4 \quad 2.3 \quad 2.1 \quad 2 \quad 1.9) \cdot in$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$$u_{5.} := \frac{f_y}{0.85 \cdot f'_c} \qquad u_{5.} = 15.7 \qquad x_{5.5} := 0.85 \cdot 0.9 \cdot f'_c \cdot t_w \cdot w_{5f.1} \cdot 2 \qquad x_{5.5} = \begin{pmatrix} 918 \\ 1028.2 \\ 1101.6 \end{pmatrix} \cdot kip$$

$$\rho_{5.8} := \frac{A_{s.4a}}{\max(w_{5f.1}) \cdot d_{5.4}} \cdot ft \qquad \rho_{5.8} = 0.0015$$

$$\phi P_{5.6} := x_{5.5} \cdot \left[-\rho_{5.8} \cdot u_{5.} + 1 - \frac{\max(e`_{5.6})}{d_{5.4}} + \sqrt{\left(1 - \frac{\max(e`_{5.6})}{d_{5.4}}\right)^2 + 2 \cdot \frac{\max(e`_{5.6}) \cdot \rho_{5.8} \cdot u_{5.}}{d_{5.4}}} \right] \qquad \phi P_{5.6} = \begin{pmatrix} 27.8 \\ 31.1 \\ 33.3 \end{pmatrix} \cdot kip$$

$$chk_{5.10} := \text{if}(\max(P_{5.8}) \le \min(\phi P_{5.6}), \text{"OK"}, \text{"NG"}) \qquad chk_{5.10} = \text{"OK"}$$

d. SIDE WALL

End Wall has less area to pick up wind for lateral force. Both the end and side walls are reinforced with the same bar sizes and patterens, there for if end wall is "OK" then the side wall is "OK"

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6) Foundations

a. Determine Loads for Foundation Design -

$$wgt_{flr} = \begin{pmatrix} 41.1 \\ 40.4 \\ 39.8 \\ 39.4 \\ 39.1 \\ 38.8 \\ 38.6 \\ 37.8 \\ 37.7 \end{pmatrix} \cdot psf \qquad wgt_{flr.ref} = \begin{pmatrix} \text{"Min Flr Uniform Weight, for L = 12 ' "} \\ \text{"Min Flr Uniform Weight, for L = 14 ' "} \\ \text{"Min Flr Uniform Weight, for L = 16 ' "} \\ \text{"Min Flr Uniform Weight, for L = 18 ' "} \\ \text{"Min Flr Uniform Weight, for L = 20 ' "} \\ \text{"Min Flr Uniform Weight, for L = 22 ' "} \\ \text{"Min Flr Uniform Weight, for L = 24 ' "} \\ \text{"Min Flr Uniform Weight, for L = 26 ' "} \\ \text{"Min Flr Uniform Weight, for L = 28 ' "} \end{pmatrix}$$

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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$$wg_{f.mx} = \begin{pmatrix} 45.1 \\ 44.4 \\ 43.8 \\ 43.4 \\ 43.1 \\ 42.8 \\ 42.6 \\ 41.8 \\ 41.7 \end{pmatrix} \cdot psf \qquad wg_{f.mx.ref} = \text{"Max Flr Unif. Wgt"}$$

$w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ $wgt_3 = 39.33 \cdot psf$ $wgt_{3.ref} = \text{"Weight of Wall Panel, Area"}$

$$wgt_{wall.plf} = \begin{pmatrix} 354 \\ 393.3 \\ 419.6 \end{pmatrix} \cdot plf \quad \text{Wall Weight} \qquad w_{misc.w} \cdot (H_{wall} - 7.5in) = \begin{pmatrix} 50.3 \\ 56.3 \\ 60.3 \end{pmatrix} \cdot plf \quad \text{Wall Misc. Weight}$$

$DL_{r.mx} = 53.7 \cdot psf$ $DL_{r.mx.ref} = \text{"Max. Roof DL"}$ $DL_{r.mn} = 46.7 \cdot psf$ $DL_{r.mn.ref} = \text{"Min. Roof DL"}$

$w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$ $p_f = 31.8 \cdot psf$ $p_{f.ref} = \text{"Flat Roof Snow Load"}$

$P_{wind_3} = -64.71 \cdot psf$ $C_{p.ref_3} = \text{"MWFRS, Ext Pres Coeff for Roof, Fig (27.3-1)"}$

$W_{lat} = 55.6 \cdot psf$ Maximum Lateral Wind Pressure (Windward + Leeward Pressures)

$C_s = 0.24$ $C_{s.ref} = \text{"Seismic Response Coeff., Eqn (12.8-2)"}$

$E_{v.S8} := 0.2 \cdot S_{DS}$ $E_{v.S8} = 0.192$ $E_{v.S8.ref} := \text{"Vertical Seismic Response Coeff."}$

$A_r := W_r \cdot L_r$ $A_f := W_f \cdot L_f$ $L_{wall} := 2 \cdot (W_{ew} + L_f)$

$A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\ ft^2$

$A_f^T = (123.74 \quad 145.65 \quad 167.57 \quad 189.49 \quad 211.4 \quad 233.32 \quad 255.24 \quad 277.15 \quad 299.07)\ ft^2$

$L_{wall}^T = (45.92 \quad 49.92 \quad 53.92 \quad 57.92 \quad 61.92 \quad 65.92 \quad 69.92 \quad 73.92 \quad 77.92)\ ft$

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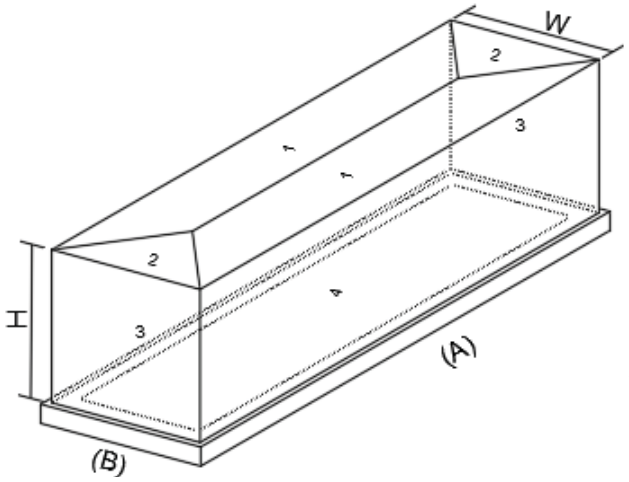

$$DL_{A.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) \ldots + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{A.EQ.f0.mn} := \frac{W_r}{2}\cdot\left(DL_{r.mn}\right) + \min\left(wgt_{wall.plf}\right) + \frac{W_f}{2}\cdot\left(wgt_{flr}\right) \ldots + \min\left(wgt_{wall.plf}\right)\cdot\frac{W_{ew}}{L_f}$$

$$DL_{B.EQ.f0.mn} := \frac{W_r}{6}\cdot\left(DL_{r.mn}\right) + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wgt_{flr}\right)$$

$$DL_{A.f0.mx} := \frac{W_r}{2}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{B.f0.mx} := \frac{W_r}{6}\cdot DL_{r.mx} + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot wg_{f.mx}$$

$$DL_{A.EQ.f0.mx} := \frac{W_r}{2}\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{2}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$DL_{B.EQ.f0.mx} := \frac{W_r}{6}\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right) + \max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right] + \frac{W_f}{6}\cdot\left(wg_{f.mx} + \frac{w_{llf.0}}{4}\right)$$

$$wgt_{z.mx} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx}\right)\right] + \max\left[\max\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\right]\cdot\max\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mx}\right)\right]\end{bmatrix} \qquad wgt_{z.mx} = 39.44\cdot kip$$

$$wgt_{z.mn} := \begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mn}\right)\right] + \min\left[wgt_{wall.plf} + w_{misc.w}\cdot\left(H_{wall} - 7.5in\right)\right]\cdot\min\left(H_{wall}\right) \ldots \\ + \max\left[A_f\cdot\left(DL_{r.mn}\right)\right]\end{bmatrix} \qquad wgt_{z.mn} = 33.49\cdot kip$$

$$z_{mx} := \max\left[\frac{H_E}{2}, \frac{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right)\cdot\left(\max\left(H_{wall}\right) + t_{r.a}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right]\cdot\frac{2}{3}\cdot t_{floor} \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\frac{\max\left(H_{wall}\right)^2}{2}\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}{\begin{bmatrix}\max\left[A_r\cdot\left(DL_{r.mx} + \frac{w_{llr.2}}{4}\right)\right] + \max\left[A_f\cdot\left(wg_{f.mx} + \frac{w_{llf.1}}{4}\right)\right] \ldots \\ + \max\left[\max\left(wgt_{wall.psf} + w_{misc.w}\right)\cdot\max\left(H_{wall}\right)\cdot 2\cdot\left(W_{sw} + W_{ew}\right)\right]\end{bmatrix}}\right] \qquad \begin{aligned} z_{mx} &= 66.88\cdot in \\ z_{mx} &= 5.57\cdot ft\end{aligned}$$

$ftg_{exp} := 36in$ — Max Height of Exposed Footing

$ftg_{depth} := 1.5ft$ — Min.Depth of ftg. Below Grade

$Arm_{top} := \max\left(H_E\right) + ftg_{exp} + ftg_{depth}$ $\quad Arm_{top} = 15.65\,ft$ — Moment Arm for Roof Level

$Arm_{bot} := ftg_{exp} + ftg_{depth}$ $\quad Arm_{bot} = 4.5\,ft$ — Moment Arm for Floor Level

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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$DL_{A.f0.mx}^{T} = (1049.4 \;\; 1045.3 \;\; 1042.3 \;\; 1039.9 \;\; 1038.1 \;\; 1036.6 \;\; 1035.3 \;\; 1031 \;\; 1030.3)\cdot plf$

$DL_{A.f0.mn}^{T} = (1225.5 \;\; 1166.3 \;\; 1122.6 \;\; 1089.1 \;\; 1062.4 \;\; 1040.8 \;\; 1022.9 \;\; 1004.6 \;\; 991.9)\cdot plf$

$DL_{B.f0.mx}^{T} = (834.6 \;\; 830.5 \;\; 827.4 \;\; 825.1 \;\; 823.2 \;\; 821.7 \;\; 820.5 \;\; 816.1 \;\; 815.5)\cdot plf$

$DL_{B.f0.mn}^{T} = (1038.6 \;\; 979.5 \;\; 935.8 \;\; 902.2 \;\; 875.6 \;\; 854 \;\; 836.1 \;\; 817.7 \;\; 805.1)\cdot plf$

$DL_{A.EO.f0.mx}^{T} = (1444 \;\; 1440 \;\; 1437 \;\; 1435 \;\; 1433 \;\; 1432 \;\; 1430 \;\; 1426 \;\; 1425)\cdot plf$

$DL_{A.EO.f0.mn}^{T} = (1225 \;\; 1166 \;\; 1123 \;\; 1089 \;\; 1062 \;\; 1041 \;\; 1023 \;\; 1005 \;\; 992)\cdot plf$

$DL_{B.EO.f0.mx}^{T} = (801 \;\; 800 \;\; 799 \;\; 798 \;\; 798 \;\; 797 \;\; 797 \;\; 795 \;\; 795)\cdot plf$

$DL_{B.EQ.f0.mn}^{T} = (573 \;\; 571 \;\; 570 \;\; 570 \;\; 569 \;\; 569 \;\; 568 \;\; 567 \;\; 566)\cdot plf$

$$E_{O.A.E.f0.C} := \frac{C_s\cdot\left(DL_{A.EQ.f0.mx}\right)}{W_f}\cdot\left(Arm_{bot} + z_{mx}\right) \qquad E_{O.B.E.f0.C} := \frac{C_s\cdot\left(DL_{B.EQ.f0.mx}\right)}{L_f}\cdot\left(Arm_{bot} + z_{mx}\right)$$

$E_{O.A.E.f0.T} := -E_{O.A.E.f0.C}$ $\qquad$ $E_{O.B.E.f0.T} := -E_{O.B.E.f0.C}$

$E_{O.A.E.f0.C}^{T} = (318.6 \;\; 317.7 \;\; 317.1 \;\; 316.5 \;\; 316.1 \;\; 315.8 \;\; 315.5 \;\; 314.6 \;\; 314.4)\cdot plf$

$E_{O.A.E.f0.T}^{T} = (-318.6 \;\; -317.7 \;\; -317.1 \;\; -316.5 \;\; -316.1 \;\; -315.8 \;\; -315.5 \;\; -314.6 \;\; -314.4)\cdot plf$

$E_{O.B.E.f0.C}^{T} = (171.6 \;\; 145.5 \;\; 126.3 \;\; 111.6 \;\; 99.9 \;\; 90.5 \;\; 82.7 \;\; 76 \;\; 70.4)\cdot plf$

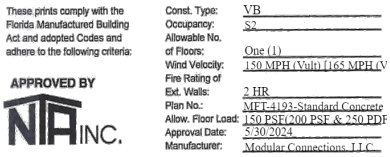
$E_{O.B.E.f0.T}^{T} = (-171.6 \;\; -145.5 \;\; -126.3 \;\; -111.6 \;\; -99.9 \;\; -90.5 \;\; -82.7 \;\; -76 \;\; -70.4)\cdot plf$

$E_{O.A.E.f1.C}^{T} = (303.5 \;\; 302.6 \;\; 302 \;\; 301.4 \;\; 301 \;\; 300.7 \;\; 300.4 \;\; 299.5 \;\; 299.3)\cdot plf$

$E_{O.A.E.f1.T}^{T} = (-303.5 \;\; -302.6 \;\; -302 \;\; -301.4 \;\; -301 \;\; -300.7 \;\; -300.4 \;\; -299.5 \;\; -299.3)\cdot plf$

$E_{O.B.E.f1.C}^{T} = (166.7 \;\; 141.3 \;\; 122.7 \;\; 108.4 \;\; 97.1 \;\; 87.9 \;\; 80.3 \;\; 73.8 \;\; 68.4)\cdot plf$

$E_{O.B.E.f1.T}^{T} = (-166.7 \;\; -141.3 \;\; -122.7 \;\; -108.4 \;\; -97.1 \;\; -87.9 \;\; -80.3 \;\; -73.8 \;\; -68.4)\cdot plf$



$E_{O.A.E.f2.C}^{T} = (288.4 \;\; 287.5 \;\; 286.8 \;\; 286.3 \;\; 285.9 \;\; 285.6 \;\; 285.3 \;\; 284.4 \;\; 284.2)\cdot plf$

$E_{O.A.E.f2.T}^{T} = (-288.4 \;\; -287.5 \;\; -286.8 \;\; -286.3 \;\; -285.9 \;\; -285.6 \;\; -285.3 \;\; -284.4 \;\; -284.2)\cdot plf$

$E_{O.B.E.f2.C}^{T} = (161.8 \;\; 137.2 \;\; 119.1 \;\; 105.2 \;\; 94.2 \;\; 85.3 \;\; 77.9 \;\; 71.6 \;\; 66.4)\cdot plf$

$E_{O.B.E.f2.T}^{T} = (-161.8 \;\; -137.2 \;\; -119.1 \;\; -105.2 \;\; -94.2 \;\; -85.3 \;\; -77.9 \;\; -71.6 \;\; -66.4)\cdot plf$

$E_{8p.A.f0} := C_s\cdot DL_{A.EQ.f0.mx} + E_{v.S8}\cdot DL_{A.EQ.f0.mx} + E_{O.A.E.f0.C}$

$E_{8p.B.f0} := C_s\cdot DL_{B.EQ.f0.mx} + E_{v.S8}\cdot DL_{B.EQ.f0.mx} + E_{O.B.E.f0.C}$

$E_{8n.A.f0} := C_s\cdot DL_{A.EQ.f0.mn} - E_{v.S8}\cdot DL_{A.EQ.f0.mn} + E_{O.A.E.f0.T}$

$E_{8n.B.f0} := C_s\cdot DL_{B.EQ.f0.mn} - E_{v.S8}\cdot DL_{B.EQ.f0.mn} + E_{O.B.E.f0.T}$

$E_{8n.A.f0}^{T} = (942.6 \;\; 939.9 \;\; 937.9 \;\; 936.4 \;\; 935.2 \;\; 934.2 \;\; 933.4 \;\; 930.6 \;\; 930.1)\cdot plf$

$E_{8n.A.f0}^{T} = (-259.8 \;\; -261.8 \;\; -263.2 \;\; -264.3 \;\; -265.1 \;\; -265.8 \;\; -266.4 \;\; -266.4 \;\; -266.8)\cdot plf$

$E_{8n.B.f0}^{T} = (517.7 \;\; 491.1 \;\; 471.5 \;\; 456.4 \;\; 444.5 \;\; 434.8 \;\; 426.8 \;\; 419.5 \;\; 413.8)\cdot plf$

$E_{8n.B.f0}^{T} = (-144.1 \;\; -118.1 \;\; -98.9 \;\; -84.2 \;\; -72.6 \;\; -63.2 \;\; -55.4 \;\; -48.8 \;\; -43.2)\cdot plf$

$E_{8n.A.f1}^{T} = (897.9 \;\; 895.2 \;\; 893.2 \;\; 891.7 \;\; 890.5 \;\; 889.5 \;\; 888.7 \;\; 885.9 \;\; 885.4)\cdot plf$

$E_{8n.B.f1}^{T} = (503 \;\; 477.1 \;\; 458 \;\; 443.3 \;\; 431.8 \;\; 422.4 \;\; 414.6 \;\; 407.5 \;\; 402)\cdot plf$

$E_{8n.A.f1}^{T} = (-240.6 \;\; -239.8 \;\; -239.3 \;\; -238.9 \;\; -238.6 \;\; -238.3 \;\; -238.1 \;\; -237.4 \;\; -237.3)\cdot plf$

$E_{8n.B.f1}^{T} = (-130.3 \;\; -105.1 \;\; -86.5 \;\; -72.2 \;\; -60.9 \;\; -51.8 \;\; -44.2 \;\; -37.8 \;\; -32.4)\cdot plf$

$E_{8n.B.f2}^{T} = (488.2 \;\; 463 \;\; 444.5 \;\; 430.3 \;\; 419 \;\; 409.9 \;\; 402.4 \;\; 395.4 \;\; 390.1)\cdot plf$

$E_{8n.A.f2}^{T} = (-228.7 \;\; -228 \;\; -227.5 \;\; -227.1 \;\; -226.8 \;\; -226.5 \;\; -226.3 \;\; -225.6 \;\; -225.4)\cdot plf$

$E_{8n.A.f2}^{T} = (853.2 \;\; 850.5 \;\; 848.5 \;\; 847 \;\; 845.8 \;\; 844.8 \;\; 844 \;\; 841.2 \;\; 840.8)\cdot plf$

$E_{8n.B.f2}^{T} = (-126.5 \;\; -102 \;\; -84 \;\; -70.1 \;\; -59.2 \;\; -50.3 \;\; -42.9 \;\; -36.7 \;\; -31.4)\cdot plf$

$LL_{r.A.} := \frac{W_r}{2}\cdot(w_{llr.2})$ $\quad LL_{r.A.} = 210\cdot plf$ $\quad LL_{r.B.} := \frac{W_r}{6}\cdot(w_{llr.2})$ $\quad LL_{r.B.} = 70\cdot plf$

$LL_{A.f0} := \frac{W_f}{2}\cdot(w_{llf.0})$ $\quad LL_{A.f0} = 1369.79\cdot plf$ $\quad LL_{B.f0} := s_{t.rib}\cdot(w_{llf.0})$ $\quad LL_{B.f0} = 500\cdot plf$

$LL_{A.f1} := \frac{W_f}{2}\cdot(w_{llf.1})$ $\quad LL_{A.f1} = 1095.83\cdot plf$ $\quad LL_{B.f1} := s_{t.rib}\cdot(w_{llf.1})$ $\quad LL_{B.f1} = 400\cdot plf$

$LL_{A.f2} := \frac{W_f}{2}\cdot(w_{llf.2})$ $\quad LL_{A.f2} = 821.88\cdot plf$ $\quad LL_{B.f2} := s_{t.rib}\cdot(w_{llf.2})$ $\quad LL_{B.f2} = 300\cdot plf$

$SL_A := \frac{W_r}{2}\cdot(p_f)$ $\quad SL_A = 190.51\cdot plf$ $\quad SL_B := \frac{W_r}{6}\cdot(p_f)$ $\quad SL_B = 63.5\cdot plf$

$W_{up.A} := \frac{W_r}{2}\cdot P_{wind_3}$ $\quad W_{up.A} = -388.24\cdot plf$ $\quad W_{up.B} := \frac{W_r}{6}\cdot P_{wind_3}$ $\quad W_{up.B} = -129.41\cdot plf$

$W_{OT.A.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{W_f}\right]$ $\quad W_{OT.A.C} = 748.4\cdot plf$

$W_{OT.B.C} := \max\left[\frac{W_{lat}\cdot H_{wall}\cdot\left(\frac{H_{wall}}{2}\right)}{L_f}\right]$ $\quad W_{OT.B.C} = 726.3\cdot plf$

$W_{OT.A.T} := -W_{OT.A.C}$ $\quad W_{OT.A.T} = -748.4\cdot plf$ $\quad W_{OT.B.T} := -W_{OT.B.C}$ $\quad W_{OT.B.T} = -726.3\cdot plf$

$W_{A.mx} := W_{up.A} + W_{OT.A.C}$ $\quad W_{A.mx} = 360.2\cdot plf$ $\quad W_{B.mx} := W_{up.B} + W_{OT.B.C}$ $\quad W_{B.mx} = 596.9\cdot plf$

$W_{A.mn} := W_{up.A} + W_{OT.A.T}$ $\quad W_{A.mn} = -1136.6\cdot plf$ $\quad W_{B.mn} := W_{up.B} + W_{OT.B.T}$ $\quad W_{B.mn} = -855.7\cdot plf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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b. Foundation Grade Beam (Perimeter Beam) Design,

consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$$Load_{f0} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f0}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f0}) \\ \min(E_{8n.A.f0}) \end{pmatrix} \qquad Load_{f0} = \begin{pmatrix} 1049.43 \\ 991.93 \\ 1369.79 \\ 0 \\ 210 \\ 190.51 \\ 360.15 \\ -1136.63 \\ 942.61 \\ -266.81 \end{pmatrix} \cdot plf$$

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f0} := asd \cdot Load_{f0} \qquad LC_{8.f0} = \begin{pmatrix} 2419 \\ 1259 \\ 2287 \\ 1266 \\ 367 \\ 2452 \\ 1329 \\ 811 \\ -87 \\ 1709 \\ 863 \\ 2715 \\ 2080 \\ 1255 \\ 408 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f0})}{\sigma_{soil}} = 13.03 \cdot in \qquad w_{8.f0.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f0})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f0})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f0.min} = 14 \cdot in$$

Width Required / Load Combination

$Fdn_{W.f0} := \max(15in, w_{8.f0.min})$ $Fdn_{W.f0} = 15 \cdot in$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.0} = 250 \cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$ Shall be $Fdn_{W.f0} = 15 \cdot in$

consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$$Load_{f1} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f1}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f1}) \\ \min(E_{8n.A.f1}) \end{pmatrix} \qquad Load_{f1} = \begin{pmatrix} 1049.43 \\ 991.93 \\ 1095.83 \\ 0 \\ 210 \\ 190.51 \\ 360.15 \\ -1136.63 \\ 897.91 \\ -240.56 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f1} := asd \cdot Load_{f1} \qquad LC_{8.f1} = \begin{pmatrix} 2145 \\ 1259 \\ 2081 \\ 1266 \\ 367 \\ 2246 \\ 1124 \\ 811 \\ -87 \\ 1678 \\ 881 \\ 2486 \\ 1888 \\ 1224 \\ 427 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f1})}{\sigma_{soil}} = 11.93 \cdot in \qquad w_{8.f1.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f1})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f1})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f1.min} = 12 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f1} := \max(14in, w_{8.f1.min}) \qquad Fdn_{W.f1} = 14 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.1} = 200 \cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$ Shall be $Fdn_{W.f1} = 14 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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consider $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$$Load_{f2} := \begin{pmatrix} \max(DL_{A.f0.mx}) \\ \min(DL_{A.f0.mn}) \\ \max(LL_{A.f2}) \\ 0 \\ \max(LL_{r.A.}, SL_A) \\ \max(SL_A) \\ \max(W_{A.mx}) \\ \min(W_{A.mn}) \\ \max(E_{8p.A.f2}) \\ \min(E_{8n.A.f2}) \end{pmatrix} \qquad Load_{f2} = \begin{pmatrix} 1049.43 \\ 991.93 \\ 821.88 \\ 0 \\ 210 \\ 190.51 \\ 360.15 \\ -1136.63 \\ 853.22 \\ -228.74 \end{pmatrix} \cdot plf$$

$$ASD = \begin{pmatrix} \text{"1a"} & \text{"(D+F)+(L+H)"} \\ \text{"2a"} & \text{"(D+F)+(max(Lr,S,R)+H)"} \\ \text{"3a"} & \text{"(D+F)+0.75(L+H)+(max(Lr,S,R)+H)"} \\ \text{"4a+"} & \text{"(D+F)+0.6(W+)"} \\ \text{"4a-"} & \text{"(D+F)+0.6(W-)"} \\ \text{"5a+"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W+)+0.5(max(Lr,S,R))"} \\ \text{"5a-"} & \text{"(D+F)+0.75(L+H)+0.75(0.6)(W-)+0.5(max(Lr,S,R))"} \\ \text{"6a+"} & \text{"0.6(D+F)+0.6(W+)"} \\ \text{"6a-"} & \text{"0.6(D+F)+0.6(W-)"} \\ \text{"7a+"} & \text{"(D+F)+0.7(E+)"} \\ \text{"7a-"} & \text{"(D+F)+0.7(E-)"} \\ \text{"8a+"} & \text{"(D+F)+0.525(E+)+0.75(L+H+S)"} \\ \text{"8a-"} & \text{"(D+F)+0.525(E-)+0.75(L+H+S)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E+)"} \\ \text{"9a+"} & \text{"0.6(D+F)+0.7(E-)"} \end{pmatrix} \qquad LC_{8.f2} := asd \cdot Load_{f2} \qquad LC_{8.f2} = \begin{pmatrix} 1871 \\ 1259 \\ 1876 \\ 1266 \\ 367 \\ 2041 \\ 918 \\ 811 \\ -87 \\ 1647 \\ 889 \\ 2257 \\ 1689 \\ 1192 \\ 435 \end{pmatrix} \cdot plf$$

$$\frac{\max(LC_{8.f2})}{\sigma_{soil}} = 10.83 \cdot in \qquad w_{8.f2.min} := ceil\left(\frac{1}{in} \cdot \max\left(\frac{\max(LC_{8.f2})}{\sigma_{soil}}, 1.5 \cdot \frac{\left|\min(0plf, LC_{8.f2})\right|}{ftg_{depth} \cdot \gamma_{NW}}\right)\right) \cdot in \qquad w_{8.f2.min} = 11 \cdot in$$

Width Required / Load Combination

$$Fdn_{W.f2} := \max(12in, w_{8.f2.min}) \qquad Fdn_{W.f2} = 12 \cdot in$$

The minimum Grade Beam (Perimeter Beam) Width for Buildings with $w_{llr.2} = 35 \cdot psf$ $w_{llr.2.ref} = \text{"Roof Live Load Option 2"}$, & $w_{llf.2} = 150 \cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$ Shall be $Fdn_{W.f2} = 12 \cdot in$

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Wind Velocity:	150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PSF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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c. Slab on Grade Foundation

1.) Criteria: use slab on grade, $t_{slab.min} := 6in$ thick, with 8" of washed gravel for foundation.

2.) Where frost is a concern extend the washed gravel down to frost depth.

3.) Since building doesn't have overturning problems, the foundation is the same as the perimeter beams for the bearing pressures. Provide a turned down edge, $t_{slab.edge} := 10in$ thick, to provide edge distance for expansion anchors. The turned down edge width will be the same as the maximum of 18" or the greates of the perimeter beam widths.

- For all loading conditions defined above the turned down edge with required is

$Fdn_{slab.thick.w} := \max(18in, Fdn_{W.f0}, Fdn_{W.f1}, Fdn_{W.f2})$

$Fdn_{slab.thick.w} = 18 \cdot in$

4.) Longitudinal Steel:

$\rho_{long.min} := 0.0018$ Minimum Longitudinal Reinforcement Ratio

$A_{s.long} := \rho_{long.min} \cdot (t_{slab.edge}) \cdot (Fdn_{slab.thick.w})$

$A_{s.long} = 0.32 \cdot in^2$ Minimum Longitudinal Area of Reinforcement

$Bar_{long} := D_4$ $Bar_{long} = \frac{1}{2} \cdot in$ Bar Diameter Size for Longitudinal Reinforcement

$Qty_{bar.long} := \max\left[2, ceil\left[\frac{A_{s.long}}{\left(\pi \cdot \frac{Bar_{long}^2}{4}\right)}\right]\right]$ Quantity of Longitudinal Reinforcement Bars

$Qty_{bar.long} = 2$

$A_{s.long.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.long}$ $A_{s.long.prov} = 0.39 \cdot in^2$

$A_{s.long.prov} = 0.39 \cdot in^2$ Area of Longitudinal Steel Provided

$chk_{8.1} := if(A_{s.long.prov} \geq A_{s.long}, "OK", "NG")$ $chk_{8.1} = "OK"$

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tem/ature Steel in Slab:

$S_{temp} := 18in$ Spacing of Tempature and Shrinkage Reinforcement

$A_{s.temp} := \rho_{long.min} \cdot t_{slab.min} \cdot S_{temp}$ $A_{s.temp} = 0.19 \cdot in^2$ Area of Tempature Reinforcement Required for Tempature and Shrinkage

$Bar_{temp} := D_4$ $Bar_{temp} = \frac{1}{2} \cdot in$ Bar Diameter Size for Tempature and Shrinkage Reinforcement

$Qty_{bar.temp} := 1$ Quantity of Tempature and Shrinkage Reinforcement Bars

$A_{s.temp.prov} := \left(\pi \cdot \frac{Bar_{long}^2}{4}\right) \cdot Qty_{bar.temp}$

$A_{s.temp.prov} = 0.2 \cdot in^2$ Area of Tempature and Shrinkage Steel Provided

$chk_{8.2} := if(A_{s.temp.prov} \geq A_{s.temp}, "OK", "NG")$ $chk_{8.2} = "OK"$

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7. CONNECTION DESIGN

a. Parameters

Yield Strength of Steel		$f_y = 60 \cdot ksi$
Compressive Strength of Concrete		$f'_c = 4500 \cdot psi$
Ultimate Tensile strength of Headed Studs		$f_{uta} = 70 \cdot ksi$
Unit weight of Roof (concrete only)		$DL_{r.mn} = 46.708 \cdot psf$
Collateral Loads on Roof		$w_{misc.r} = 7 \cdot psf$
Unit Weight of Roof		$DL_{r.mx} = 53.7 \cdot psf$
Unit weight of Wall (concrete only)		$wgt_{wall.psf} = 39.333 \cdot psf$
Collateral Loads on Walls		$w_{misc.w} = 6 \cdot psf$
Unit Weight of Wall	$\omega_{wall} := wgt_{wall.psf} + w_{misc.w}$	$\omega_{wall} = 45.3 \cdot psf$
Roof Live Load		$w_{llr} = 35 \cdot psf$

WIND LOAD PARAMETERS:

Building Design Wind Pressure	$v_{MWFRS} := W_{lat}$	$v_{MWFRS} = 55.6 \cdot psf$
Max Wall Wind Pressure	$v_{w.wall} := P_{wind_0}$	$v_{w.wall} = 43.3 \cdot psf$
Wind Pressure on Side Wall (Suction)	$v_{w.side} := \left(\left\|P_{wind_2}\right\|\right)$	$v_{w.side} = 39 \cdot psf$
Uplift Wind Pressure	$v_{up} := \left\|P_{wind_3}\right\|$	$v_{up} = 64.7 \cdot psf$
Total Wind Shear at Roof (EW)	$V_{EW\_wmaxW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wmaxW} = 3.62 \cdot kip$
	$V_{EW\_wminW} := \max\left[\left(v_{MWFRS} \cdot \frac{H_E}{2}\right) \cdot W_{ew}\right]$	$V_{EW\_wminW} = 3.618 \cdot kip$
Total Wind Shear at Roof (NS)	$V_{NS\_wmaxL} := v_{MWFRS} \cdot \max\left(\frac{H_E}{2}\right) \cdot \max(L)$	$V_{NS\_wmaxL} = 8.683 \cdot kip$
Wind Shear per Foot at either side of Roof (EW)	$v_{EW\_wmaxW} := \frac{1}{4} \cdot \frac{V_{EW\_wmaxW}}{\max(L)}$	$v_{EW\_wmaxW} = 0.032 \cdot klf$
	$v_{EW\_wminW} := \frac{1}{4} \cdot \frac{V_{EW\_wminW}}{\max(L)}$	$v_{EW\_wminW} = 0.032 \cdot klf$

** End walls are approximately square and will act as two-way plate distributing load evenly to all four sides.*

Wind Shear per Foot at either side of Roof (NS)	$v_{NS\_wmaxW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wmaxW} = 0.372 \cdot klf$
	$v_{NS\_wminW} := \frac{1}{2} \cdot \frac{V_{NS\_wmaxL}}{W_{ew}}$	$v_{NS\_wminW} = 0.372 \cdot klf$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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SEISMIC LOAD PARAMETERS: *For worst case seismic load in E-W direction, look at longest, widest shelter.*

Seismic Coefficient

$C_s = 0.24$

Weight of Roof

$WT_{r\_maxW} := \max\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_maxW} = 21.24\cdot kip$

$WT_{r\_minW} := \max\left[\left(DL_{r.mn} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r\right]$ $WT_{r\_minW} = 18.86\cdot kip$

Weight of Walls

$WT_{wall\_maxW} := \max\left[\left(\omega_{wall}\right)\cdot\left(\max\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_maxW} = 27.08\cdot kip$

$WT_{wall\_minW} := \min\left[\left(\omega_{wall}\right)\cdot\left(\min\left(H_{wall}\right)\cdot\max(2L)\right)\right]$ $WT_{wall\_minW} = 22.85\cdot kip$

Total Seismic Shear at Roof

$$V_{smaxW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_maxW} + \frac{WT_{wall\_maxW}}{2}\right)$$ $V_{smaxW\_rf} = 8.346\cdot kip$

$$V_{sminW\_rf} := \left(C_s\right)\cdot\left(WT_{r\_minW} + \frac{WT_{wall\_minW}}{2}\right)$$ $V_{sminW\_rf} = 7.267\cdot kip$

Seismic Shear per Foot at either side of Roof (EW)

$$v_{EW\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{L}\right)$$ $v_{EW\_smaxW} = 0.348\cdot klf$

$$v_{EW\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{\max(L)}\right)$$ $v_{EW\_sminW} = 0.13\cdot klf$

Seismic Shear per Foot at either side of Roof (NS)

$$v_{NS\_smaxW} := \max\left(\frac{1}{2}\cdot\frac{V_{smaxW\_rf}}{W_{ew}}\right)$$ $v_{NS\_smaxW} = 0.358\cdot klf$

$$v_{NS\_sminW} := \min\left(\frac{1}{2}\cdot\frac{V_{sminW\_rf}}{W_{ew}}\right)$$ $v_{NS\_sminW} = 0.311\cdot klf$

PLATE CONNECTION FORMULAS

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

$N_{sa} = n_{stud}\cdot A_{SE}\cdot f_{uta}$ Nominal Strength for Group of Studs in Tension as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

s2 = Spacing Between Outermost Studs Perpendicular to Edge

$$\psi_{ec\_N} = \text{if}\left(e'_N > 0, \frac{1}{1 + \frac{2\cdot e'_N}{3\cdot h_{ef}}}, 1.0\right)$$ Modification Factor for Eccentric Loading

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\cdot\psi_{ec\_N}\cdot\psi_{ed\_N}\cdot\psi_{c\_N}\cdot\psi_{cp\_N}\cdot N_b$$ Nominal Concrete Breakout Strength in Tension for Anchors

PULLOUT STRENGTH OF ANCHOR IN TENSION

A_{brg} = Bearing Area of Stud Head

ψ_{c_P} Modification Factor for Cracking

$N_p = 8\cdot A_{brg}\cdot f'_c$ Pullout Strength in Tension of Single-Headed Stud

$N_{pn} = \psi_{c\_P}\cdot N_p\cdot n$ Nominal Pullout Strength in Tension

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Occupancy:	S2
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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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SHEAR CAPACITIES:

STEEL SHEAR STRENGTH OF ANCHOR IN SHEAR

n_{stud} = Number of Studs

d_0 = Diameter of Stud(s)

A_{SE} = Cross Sectional Area of Studs

f_{uta} = Specified Tensile Strength of Anchor

$V_{sa} = n_{stud} \cdot A_{SE} \cdot f_{uta}$ Nominal Strength in Shear of Anchor or Group of Anchors as Governed by Steel Strength

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

Since the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load will be transferred to the stiffer and stronger rear anchor. Therefore, the only failure mode that need be considered is the entire group failing as a unit thus the distribution of forces indicates that the total shear would be critical on the rear anchor and its projected area.

h_{ef} = Effective Anchor Embedment Depth

c_{a1} = Distance from Center of Furthest Anchor to Edge of Concrete in Direction of Shear

c_{a2} = Distance from Center of Furthest Anchor to Edge of Concrete Perpendicular to c_{a1}

s1 = Stud Spacing

h_a = Thickness of Member into which Anchor is Embedded, Measured Parallel to Anchor Axis

h_{min} = Minimum of h_a and $1.5c_{a1}$

$l_e = \text{if}\left(h_{ef} < 8 \cdot d_0, h_{ef}, 8 \cdot d_0\right)$ Load Bearing Length of Anchor for Shear

$A_{Vc\_op} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for Out-of-Plane Shear

$A_{Vc\_ip} = \left(1.5 \cdot c_{a1} + s1 + 1.5 \cdot c_{a1}\right) \cdot \left(h_a\right)$ Projected Concrete Failure Area of Group for In-Plane Shear

$A_{Vco} = 4.5 \cdot c_{a1}^{\ 2}$ Projected Concrete Failure Area of One Anchor

e'_v = Distance Between Resultant Shear Load of a Group of Anchors and the Centroid of the Group of Anchors Loaded in Shear in the Same Direction

$\psi_{ec\_V} = \text{if}\left(e'_V > 0, \frac{1}{1 + \frac{2 \cdot e'_V}{3 \cdot c_{a1}}}, 1.0\right)$ Modification Factor for Eccentric Loading

$\psi_{ed\_V} = \text{if}\left(c_{a2} < 1.5 \cdot c_{a1}, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}, 1.0\right)$ Modification Factor (Shear) for Edge Effects

ψ_{c_V} Modification Factor for Cracking

$V_b = 8 \cdot \left(\frac{l_e}{d_0}\right)^{0.2} \cdot \sqrt{d_0} \cdot \left(\sqrt{f'_c \cdot \text{psi}}\right) \cdot c_{a1}^{\ 1.5}$ Basic Breakout Strength

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot V_b\right)$ Nominal Concrete Breakout Strength in Shear of a Group of Anchors

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$c_{a\_min} = \min\left(\left(c_{a1} \quad c_{a2}\right)\right)$ Minimum Distance From Center of Anchor to Edge of Concrete

$k_{cp} = \text{if}\left(h_{ef} < 2.5 \cdot \text{in}, 1.0, 2.0\right)$ Coefficient for Pryout Strength

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k_c — Coefficient for Basic Concrete Breakout Strength in Tension

$A_{Nco} = 9 \cdot h_{ef}^{2}$ — Projected Concrete Failure Area of a Single Anchor

$A_{Nc} = \left(b_{pl\_3} + 1.5 \cdot h_{ef} \cdot 2\right) \cdot \left(1.5 \cdot h_{ef} \cdot 2\right)$ — Projected Concrete Failure Area Group of Anchors

$\psi_{ed\_N} = \text{if}\left(c_{a\_min} > 1.5 \cdot h_{ef}, 1.0, 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h_{ef}}\right)$ — Modification Factor for Tensile Strength of Anchors Based on Edge Effects

ψ_{c_N} — Modification Factor For Tensile Strength Due to Cracks

ψ_{cp_N} — Modification Factor For Tensile Strength for Post Installed Anchors

$N_b = \left(k_c\right) \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot h_{ef}^{1.5}$ — Basic Breakout Strength in Tension of a single Anchor in Tension

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ — Nominal Concrete Breakout Strength in Tension of a Single Anchor

$V_{cp} = k_{cp} \cdot N_{cb}$ — Nominal Concrete Pryout Strength

PLATE SUMMARY:

Shear Allowable Summary

$\begin{pmatrix} 0.65V_{sa} \\ 0.75 \cdot V1_{cbg} \\ 0.7V_{cp} \end{pmatrix} =$ ductile steel failure in shear; concrete breakout in shear; concrete pryout in shear

Controlling tensile capacity

$\phi P_{pl3} = \min\left(\left(0.75 \cdot N_{sa} \quad 0.75 \cdot N_{cbg} \quad 0.7 \cdot N_{pn}\right)\right)$

Plate Summary

$\frac{Tu_{po}}{\phi P_{pl}}$ or $\frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.0$

if both are > 0.2, then $\frac{Tu_{po}}{\phi P_{pl}} + \frac{Vu_{op}}{\left(0.75 \cdot V1_{cbg}\right)} + \frac{Vu_{ip}}{\left(0.75 \cdot V2_{cbg}\right)} \le 1.2$

****

b. Roof Panel-To-End Wall

$A_{t\_r.REW} := 2 \cdot \left(\frac{1}{2} \frac{W_r}{2} \cdot \frac{W_r}{2}\right)$ $A_{t\_r.REW} = 36\,ft^2$ — Max Tributary Triangular Area of Roof for End Wall

$v_{o\_s.REW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{2}\right)$ $v_{o\_s.REW} = 0.058 \cdot klf$ — Seismic load along end wall into roof diaphragm

$v_{i\_s.REW} := v_{NS\_smaxW}$ $v_{i\_s.REW} = 0.358 \cdot klf$ — Seismic load in NS direction from roof diaphragm

$t_{up.REW} := \left(v_{up}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{up.REW} = 0.194 \cdot klf$ — Uplift load from wind on roof

$t_{DL.REW} := \left(DL_{r.mx} + w_{misc.r}\right) \cdot \left(\frac{A_{t\_r.REW}}{W_r}\right)$ $t_{DL.REW} = 0.182 \cdot klf$ — Total Dead Load on roof

$v_{o\_w.REW} := \frac{v_{w.side} \cdot W_{ew}}{4}$ $v_{o\_w.REW} = 0.114 \cdot klf$ — Wind load along end wall

$v_{i\_w.REW} := v_{NS\_wmaxW}$ $v_{i\_w.REW} = 0.372 \cdot klf$ — Direct wind load in NS direction

$Plate_{7.1}$ = "#2" at underside of roof panel on end wall

$Plate_{7.2}$ = "#49" at top of end wall

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Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

No. of Plates & maximum plate spacing $No_{pl.REW} = 7$

$s_{REW} := \frac{W_{ew} - 2\cdot(1ft + 2in)}{No_{pl.REW} - 1}$ $s_{REW} = 18.67\cdot in$

$s_{EWR} := \frac{W_{ew} - 2\cdot(1ft)}{No_{pl.REW} - 1}$ $s_{EWR} = 19.33\cdot in$

$e_{PL.REW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.1} := if\left(s_{L.7.1} = 0in, s_{w.7.1}, if\left(s_{w.7.1} = 0in, s_{L.7.1}, min\left(s_{L.7.1}, s_{w.7.1}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.1} := 0.5 + \frac{0.5s_{7.1} + 2e_{PL.REW}}{s_{7.1}}$ $F_{7.1} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.1

$s_{7.2} := if\left(s_{L.7.2} = 0in, s_{w.7.2}, if\left(s_{w.7.2} = 0in, s_{L.7.2}, min\left(s_{L.7.2}, s_{w.7.2}\right)\right)\right) = 3\cdot in$ $s_{7.1} = 3\cdot in$

$F_{7.2} := 0.5 + \frac{0.5s_{7.2} + 2e_{PL.REW}}{s_{7.2}}$ $F_{7.2} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.2

Factored loads - Roof - Endwall (Roof Plates)

$Vu_{o.s.REW} := F_{7.1}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{REW}\right)$ $Vu_{o.s.REW} = 0.032\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.REW} := F_{7.1}\cdot 1.0v_{i\_s.REW}\cdot s_{REW}$ $Vu_{i.s.REW} = 0.649\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.REW} := F_{7.1}\cdot 1.0v_{o\_w.REW}\cdot s_{REW}$ $Vu_{o.w.REW} = 0.207\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.REW} := F_{7.1}\cdot 1.0v_{i\_w.REW}\cdot s_{REW}$ $Vu_{i.w.REW} = 0.675\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.REW} := F_{7.1}\cdot max\left[\left(1.0\cdot t_{up.REW} - 0.9\cdot t_{DL.REW}\right)\cdot s_{REW}, 0\cdot kip\right]$

$Tu_{T.REW} = 0.055\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Endwall (End Wall Plates)

$Vu_{o.s.EWR} := F_{7.2}\cdot 0.3\left(v_{o\_s.REW}\cdot s_{EWR}\right)$ $Vu_{o.s.EWR} = 0.033\cdot kip$ 0.3E out-of-plane V, seismic

$Vu_{i.s.EWR} := F_{7.2}\cdot 1.0v_{i\_s.REW}\cdot s_{EWR}$ $Vu_{i.s.EWR} = 0.672\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{o.w.EWR} := F_{7.2}\cdot max\left(1.0v_{o\_w.REW}\cdot s_{EWR}, 0kip\right)$ $Vu_{o.w.EWR} = 0.214\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.EWR} := F_{7.2}\cdot 1.0v_{i\_w.REW}\cdot s_{EWR}$ $Vu_{i.w.EWR} = 0.699\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.EWR} := F_{7.2}\cdot 1.0v_{o\_w.REW}\cdot s_{EWR}$ $Tu_{T.EWR} = 0.214\cdot kip$ 0.9DL + 1.0W T, roof uplift

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Plate Calcs Roof to Endwall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.1} = 5\cdot in$ $w_{pl.7.1} = 5\cdot in$ $x_{Le.7.1} = 1\cdot in$ $x_{we.7.1} = 2.5\cdot in$ $t_{pl.7.1} = 0.313\cdot in$

$h_{ef.7.1} = 1.719\cdot in$ $h_{a.7.1} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.1} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.1} = 4\cdot in$

Shear Studs: $n_{stud.7.1} = 2$ $d_{0.7.1} = 0.5\cdot in$ $A_{SE.7.1} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.1} = n_{stud.7.1}\cdot A_{SE.7.1}\cdot f_{uta}$ $N_{sa.7.1} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.1} = 1.719\cdot in$ $c_{a1\_po.7.1} = 10.5\cdot in$ $c_{a2\_po.7.1} = 6.5\cdot in$ $s1_{7.1} = 3\cdot in$ $s2_{7.1} = 0\cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.1} = 24$ $A_{Nc.7.1} = 42.07 \cdot in^2$ $A_{Nco.7.1} = 26.59 \cdot in^2$ $e'_{N.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.1} = 1.00$ $\psi_{ed\_N.7.1} = 1.00$ $\psi_{c\_N.7.1} = 1.00$ $\psi_{cp\_N.7.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.1} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.1} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.1} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.1} = n_{stud.7.1} \cdot A_{SE.7.1} \cdot f_{uta}$ $V_{sa.7.1} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.1} = 1.719 \cdot in$ $h_{min.7.1} = 4 \cdot in$ $c_{a1\_op.7.1} = 6.5 \cdot in$ $c_{a2\_op.7.1} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.1} = 90 \cdot in^2$ $A_{Vco\_op.7.1} = 190 \cdot in^2$ $e'_{V.7.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.1} = 1.00$ $\psi_{ed\_Vop.7.1} = 1.00$ $\psi_{c\_Vop.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.1} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.1} = 4.57 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.1a} = 0 \cdot in$ $s2_{7.1a} = 3 \cdot in$ $l_{e.7.1} = 1.719 \cdot in$ $h_{ef.7.1} = 1.719 \cdot in$ $h_{a.7.1} = 4 \cdot in$ $h_{min.7.1a} = 4 \cdot in$

$c_{a1\_ip.7.1} = 13.5 \cdot in$ $c_{a2\_ip.7.1} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.1} = 107 \cdot in^2$ $A_{Vco\_ip.7.1} = 820 \cdot in^2$ $e'_{V.7.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.1} = 1.00$ $\psi_{ed\_Vip.7.1} = 0.80$ $\psi_{c\_Vip.7.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.1} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.1} = 3 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.1} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.1} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.1} = k_{cp.7.1} \cdot N_{cbg.7.1}$ $V_{cpg.7.1} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.1} \\ 0.75 \cdot V_{cbg\_op.7.1} \\ 0.75 \cdot V_{cbg\_ip.7.1} \\ 0.7V_{cpg.7.1} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.REW} = \min(0.75 \cdot N_{sa.7.1} \quad 0.75 \cdot N_{cbg.7.1} \quad 0.7 \cdot N_{pn.7.1})$ $\phi P_{pl.REW} = 4.305 \cdot kip$

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Plate Summary $\frac{Tu_{T.REW}}{\phi P_{pl.REW}} = 0.013$ or $\left[\frac{Vu_{o.s.REW}}{0.75 \cdot V_{cbg\_op.7.1}} + \frac{Vu_{i.s.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.297 < 1.0$

or $\left[\frac{Vu_{o.w.REW}}{\left(0.75 \cdot V_{cbg\_op.7.1}\right)} + \frac{Vu_{i.w.REW}}{\left(0.75 \cdot V_{cbg\_ip.7.1}\right)}\right] = 0.36 < 1.0$

$Result_{REW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.REW}$ = "not required" < 1.2

$chk_{6.REW}$ = "OK"

Plate Calcs Roof to Endwall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.2} = 6.5 \cdot in$ $w_{pl.7.2} = 6.5 \cdot in$ $x_{Le.7.2} = 1.75 \cdot in$ $x_{we.7.2} = 1.75 \cdot in$ $t_{pl.7.2} = 0.313 \cdot in$

$h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.2} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.2} = 4 \cdot in$

Shear Studs: $n_{stud.7.2} = 4$ $d_{0.7.2} = 0.5 \cdot in$ $A_{SE.7.2} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $N_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.2} = 1.719 \cdot in$ $c_{a1\_po.7.2} = 10.5 \cdot in$ $c_{a2\_po.7.2} = 5.75 \cdot in$ $s1_{7.2} = 3 \cdot in$ $s2_{7.2} = 3 \cdot in$

Design Factors: $k_{c.7.2} = 24$ $A_{Nc.7.2} = 66.54 \cdot in^2$ $A_{Nco.7.2} = 26.59 \cdot in^2$ $e'_{N.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.2} = 1.00$ $\psi_{ed\_N.7.2} = 1.00$ $\psi_{c\_N.7.2} = 1.00$ $\psi_{cp\_N.7.2} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.2} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.2} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.2} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.2} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.2} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.2} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.2} = n_{stud.7.2} \cdot A_{SE.7.2} \cdot f_{uta}$ $V_{sa.7.2} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.2} = 1.719 \cdot in$ $h_{min.7.2} = 4 \cdot in$ $c_{a1\_op.7.2} = 8.75 \cdot in$ $c_{a2\_op.7.2} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.2} = 107 \cdot in^2$ $A_{Vco\_op.7.2} = 345 \cdot in^2$ $e'_{V.7.2} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.2} = 1.00$ $\psi_{ed\_Vop.7.2} = 0.94$ $\psi_{c\_Vop.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.2} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.2} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.2a} = 3 \cdot in$ $s2_{7.2a} = 3 \cdot in$ $l_{e.7.2} = 1.719 \cdot in$ $h_{ef.7.2} = 1.719 \cdot in$ $h_{a.7.2} = 4 \cdot in$ $h_{min.7.2a} = 4 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$c_{a1\_ip.7.2} = 13.5 \cdot in$ $c_{a2\_ip.7.2} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.2} = 116 \cdot in^2$ $A_{Vco\_ip.7.2} = 820 \cdot in^2$ $e'_{V.7.2a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.2} = 1.00$ $\psi_{ed\_Vip.7.2} = 0.79$ $\psi_{c\_Vip.7.2} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.2} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.2} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Design Factors: $k_{cp.7.2} = 1.00$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.2} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.2} = k_{cp.7.2} \cdot N_{cbg.7.2}$ $V_{cpg.7.2} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.2} \\ 0.75 \cdot V_{cbg\_op.7.2} \\ 0.75 \cdot V_{cbg\_ip.7.2} \\ 0.7V_{cpg.7.2} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWR} = \min\left(\left(0.75 \cdot N_{sa.7.2} \quad 0.75 \cdot N_{cbg.7.2} \quad 0.7 \cdot N_{pn.7.2}\right)\right)$ $\phi P_{pl.EWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Tu_{T.EWR}}{\phi P_{pl.EWR}} = 0.031$ or $\left[\dfrac{Vu_{o.s.EWR}}{0.75 \cdot V_{cbg\_op.7.2}} + \dfrac{Vu_{i.s.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.289 < 1.0$

or $\left[\dfrac{Vu_{o.w.EWR}}{\left(0.75 \cdot V_{cbg\_op.7.2}\right)} + \dfrac{Vu_{i.w.EWR}}{\left(0.75 \cdot V_{cbg\_ip.7.2}\right)}\right] = 0.355 < 1.0$

$Result_{EWR} = "< 0.2, full strength shear/tension permitted"$ $IC_{pl.EWR} = "not required"$ < 1.2

$chk_{6.EWR} = "OK"$

Weld Between $Plate_{7.1} = "\#2"$ and $Plate_{7.2} = "\#49"$ along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.REW.1} := \frac{\left(\sqrt{Vu_{o.s.EWR}^2 + Vu_{i.s.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.1} = 0.197 \cdot in$$

$$l_{w\_Rqrd.REW.2} := \frac{\left(\sqrt{Tu_{T.EWR}^2 + Vu_{o.w.EWR}^2 + Vu_{i.w.EWR}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.REW.2} = 0.223 \cdot in$$

$$l_{w\_Rqrd.REW.3} := \frac{\min\left(w_{pl.7.2}, w_{pl.7.1}\right)}{2} \qquad l_{w\_Rqrd.REW.3} = 2.5 \cdot in$$

$l_{w\_reqd.REW} := \max\left(l_{w\_Rqrd.REW.1}, l_{w\_Rqrd.REW.2}, l_{w\_Rqrd.REW.3}\right)$ $l_{w\_reqd.REW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

c. Roof Panel-To-Side Wall

$w_{t.r.RSW} := \frac{W_r}{2}$ $w_{t.r.RSW} = 6\ ft$ Tributary Width, Roof for Side Wall

$v_{o\_s.RSW} := C_s\left(\omega_{wall}\cdot\frac{\max(H_{wall})}{2}\right)$ $v_{o\_s.RSW} = 0.058\cdot klf$ Seismic load along sidewall into roof diaphragm

$v_{i\_s.RSW} := v_{EW\_smaxW}$ $v_{i\_s.RSW} = 0.348\cdot klf$ Seismic load in EW direction from roof diaphragm

$t_{up.RSW} := v_{up}\cdot(w_{t.r.RSW})$ $t_{up.RSW} = 0.388\cdot klf$ Uplift load from wind on roof

$t_{DL.RSW} := (DL_{r.mx})\cdot(w_{t.r.RSW})$ $t_{DL.RSW} = 0.322\cdot klf$ Total Dead Load on roof

$v_{o\_w.RSW} := \frac{v_{w.side}\cdot W_{ew}}{2}$ $v_{o\_w.RSW} = 0.228\cdot klf$ Wind load along sidewall

$v_{ip\_w.RSW} := v_{EW\_wmaxW}$ $v_{ip\_w.RSW} = 0.032\cdot klf$ Direct wind load in EW direction - conservatively use min(L)

$Plate_{7.3}$ = "#2" at underside of roof panel on side wall $Plate_{7.4}$ = "#49" at top of side wall

Plate resists in-plane shear load from roof diaphragm, out-of- plane shear from wall suction, & pullout from roof uplift.

Maximum plate spacing $s_{max.RSW} = 48\cdot in$ Minimum # of Plates = 4

$e_{PL.RSW} := 0.25in$ Allowance for plate mis-alignment $s_{REW} = 18.67\cdot in$ $s_{EWR} = 19.33\cdot in$

$s_{7.3} := \text{if}(s_{L.7.3} = 0in, s_{w.7.3}, \text{if}(s_{w.7.3} = 0in, s_{L.7.3}, \min(s_{L.7.3}, s_{w.7.3}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.3} := 0.5 + \frac{0.5s_{7.3} + 2e_{PL.RSW}}{s_{7.3}}$ $F_{7.3} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.3

$s_{7.4} := \text{if}(s_{L.7.4} = 0in, s_{w.7.4}, \text{if}(s_{w.7.4} = 0in, s_{L.7.4}, \min(s_{L.7.4}, s_{w.7.4}))) = 3\cdot in$ $s_{7.3} = 3\cdot in$

$F_{7.4} := 0.5 + \frac{0.5s_{7.4} + 2e_{PL.RSW}}{s_{7.4}}$ $F_{7.4} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.4

Factored loads - Roof - Side Wall (Roof Plates)

$Vu_{o.s.RSW} := F_{7.3}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.RSW} = 0.271\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.RSW} := F_{7.3}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.RSW} = 0.487\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Vu_{o.w.RSW} := F_{7.3}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Vu_{o.w.RSW} = 1.062\cdot kip$ 1.0W out-of-plane V, wall suction

$Vu_{i.w.RSW} := F_{7.3}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.RSW} = 0.151\cdot kip$ 1.0W in-plane V roof diaphragm

$Tu_{T.RSW} := F_{7.3}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Tu_{T.RSW} = 0.458\cdot kip$ 0.9DL + 1.0W T, roof uplift

Factored loads - Roof - Side Wall (Side Wall Plates)

$Vu_{o.s.SWR} := F_{7.4}\cdot 1.0(v_{o\_s.RSW}\cdot s_{max.RSW})$ $Vu_{o.s.SWR} = 0.271\cdot kip$ 1.0E out-of-plane V, seismic

$Vu_{i.s.SWR} := F_{7.4}\cdot 0.3v_{i\_s.RSW}\cdot s_{max.RSW}$ $Vu_{i.s.SWR} = 0.487\cdot kip$ 0.3E in-plane V, seismic roof diaphragm

$Tu_{o.w.SWR} := F_{7.4}\cdot 1.0v_{o\_w.RSW}\cdot s_{max.RSW}$ $Tu_{o.w.SWR} = 1.062\cdot kip$ 1.0W T, wall suction

$Vu_{i.w.SWR} := F_{7.4}\cdot 1.0v_{ip\_w.RSW}\cdot s_{max.RSW}$ $Vu_{i.w.SWR} = 0.151\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{T.SWR} := F_{7.4}\cdot\max\left[\left(1.0\cdot t_{up.RSW} - 0.9\cdot t_{DL.RSW}\right)\cdot s_{max.RSW}, 0\cdot kip\right]$

$Vu_{T.SWR} = 0.458\cdot kip$ 0.9DL + 1.0W out-of-plane V, roof uplift

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1090 Industrial Blvd.
Bessemer, AL 35022
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Email: info@modularconnections.com

Plate Calcs Roof to Sidewall - Roof Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.3} = 5 \cdot in$ $w_{pl.7.3} = 5 \cdot in$ $x_{Le.7.3} = 1 \cdot in$ $x_{we.7.3} = 2.5 \cdot in$ $t_{pl.7.3} = 0.313 \cdot in$

$h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.3} = 22 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.3} = 2.25 \cdot in$

Shear Studs: $n_{stud.7.3} = 2$ $d_{0.7.3} = 0.5 \cdot in$ $A_{SE.7.3} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $N_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.3} = 1.719 \cdot in$ $c_{a1\_po.7.3} = 20.5 \cdot in$ $c_{a2\_po.7.3} = 4.75 \cdot in$ $s1_{7.3} = 3 \cdot in$ $s2_{7.3} = 0 \cdot in$

Design Factors: $k_{c.7.3} = 24$ $A_{Nc.7.3} = 42.07 \cdot in^2$ $A_{Nco.7.3} = 26.59 \cdot in^2$ $e'_{N.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.3} = 1.00$ $\psi_{ed\_N.7.3} = 1.00$ $\psi_{c\_N.7.3} = 1.00$ $\psi_{cp\_N.7.3} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.3} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.3} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.3} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.3} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.3} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.3} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.3} = n_{stud.7.3} \cdot A_{SE.7.3} \cdot f_{uta}$ $V_{sa.7.3} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.3} = 1.719 \cdot in$ $h_{min.7.3} = 4 \cdot in$ $c_{a1\_op.7.3} = 4.75 \cdot in$ $c_{a2\_op.7.3} = 20.5 \cdot in$

Design Factors: $A_{Vc\_op.7.3} = 69 \cdot in^2$ $A_{Vco\_op.7.3} = 102 \cdot in^2$ $e'_{V.7.3} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.3} = 1.00$ $\psi_{ed\_Vop.7.3} = 1.00$ $\psi_{c\_Vop.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.3} = 5.03 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.3} = 4.1 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.3a} = 0 \cdot in$ $s2_{7.3a} = 3 \cdot in$ $l_{e.7.3} = 1.719 \cdot in$ $h_{ef.7.3} = 1.719 \cdot in$ $h_{a.7.3} = 4 \cdot in$ $h_{min.7.3a} = 4 \cdot in$

$c_{a1\_ip.7.3} = 23.5 \cdot in$ $c_{a2\_ip.7.3} = 4.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.3} = 160 \cdot in^2$ $A_{Vco\_ip.7.3} = 2485 \cdot in^2$ $e'_{V.7.3a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.3} = 1.00$ $\psi_{ed\_Vip.7.3} = 0.74$ $\psi_{c\_Vip.7.3} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.3} = 55.34 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.3} = 3.17 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.3} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.3} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.3} = k_{cp.7.3} \cdot N_{cbg.7.3}$ $V_{cpg.7.3} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.3} \\ 0.75 \cdot V_{cbg\_op.7.3} \\ 0.75 \cdot V_{cbg\_ip.7.3} \\ 0.7V_{cpg.7.3} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.076 \\ 2.374 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.RSW} = \min\left(\left(0.75 \cdot N_{sa.7.3} \quad 0.75 \cdot N_{cbg.7.3} \quad 0.7 \cdot N_{pn.7.3}\right)\right)$ $\phi P_{pl.RSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{T.RSW}}{\phi P_{pl.RSW}} = 0.106$ or $\left[\dfrac{Vu_{o.s.RSW}}{0.75 \cdot V_{cbg\_op.7.3}} + \dfrac{Vu_{i.s.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.293 < 1.0$

or $\left[\dfrac{Vu_{o.w.RSW}}{\left(0.75 \cdot V_{cbg\_op.7.3}\right)} + \dfrac{Vu_{i.w.RSW}}{\left(0.75 \cdot V_{cbg\_ip.7.3}\right)}\right] = 0.409 < 1.0$

$Result_{RSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.RSW}$ = "not required" < 1.2

$chk_{6.RSW}$ = "OK"

Plate Calcs Roof to Side Wall - Side Wall Plates

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.4} = 6.5 \cdot in$ $w_{pl.7.4} = 6.5 \cdot in$ $x_{Le.7.4} = 1.75 \cdot in$ $x_{we.7.4} = 1.75 \cdot in$ $t_{pl.7.4} = 0.313 \cdot in$

$h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.4} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.4} = 4 \cdot in$

Shear Studs: $n_{stud.7.4} = 4$ $d_{0.7.4} = 0.5 \cdot in$ $A_{SE.7.4} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $N_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.4} = 1.719 \cdot in$ $c_{a1\_po.7.4} = 10.5 \cdot in$ $c_{a2\_po.7.4} = 5.75 \cdot in$ $s1_{7.4} = 3 \cdot in$ $s2_{7.4} = 3 \cdot in$

Design Factors: $k_{c.7.4} = 24$ $A_{Nc.7.4} = 66.54 \cdot in^2$ $A_{Nco.7.4} = 26.59 \cdot in^2$ $e'_{N.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.4} = 1.00$ $\psi_{ed\_N.7.4} = 1.00$ $\psi_{c\_N.7.4} = 1.00$ $\psi_{cp\_N.7.4} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.4} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.4} = 9.08 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.4} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.4} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.4} = 28.27 \cdot kip$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.4} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.4} = n_{stud.7.4} \cdot A_{SE.7.4} \cdot f_{uta}$ $V_{sa.7.4} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.4} = 1.719 \cdot in$ $h_{min.7.4} = 4 \cdot in$ $c_{a1\_op.7.4} = 8.75 \cdot in$ $c_{a2\_op.7.4} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.4} = 107 \cdot in^2$ $A_{Vco\_op.7.4} = 345 \cdot in^2$ $e'_{V.7.4} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.4} = 1.00$ $\psi_{ed\_Vop.7.4} = 0.94$ $\psi_{c\_Vop.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.4} = 12.57 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.4} = 4.38 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.4a} = 3 \cdot in$ $s2_{7.4a} = 3 \cdot in$ $l_{e.7.4} = 1.719 \cdot in$ $h_{ef.7.4} = 1.719 \cdot in$ $h_{a.7.4} = 4 \cdot in$ $h_{min.7.4a} = 4 \cdot in$

$c_{a1\_ip.7.4} = 13.5 \cdot in$ $c_{a2\_ip.7.4} = 5.75 \cdot in$

Design Factors: $A_{Vc\_ip.7.4} = 116 \cdot in^2$ $A_{Vco\_ip.7.4} = 820 \cdot in^2$ $e'_{V.7.4a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.4} = 1.00$ $\psi_{ed\_Vip.7.4} = 0.79$ $\psi_{c\_Vip.7.4} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.4} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.4} = 3.21 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.4} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.4} = 9.08 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.4} = k_{cp.7.4} \cdot N_{cbg.7.4}$ $V_{cpg.7.4} = 9.08 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.4} \\ 0.75 \cdot V_{cbg\_op.7.4} \\ 0.75 \cdot V_{cbg\_ip.7.4} \\ 0.7V_{cpg.7.4} \end{pmatrix} = \begin{pmatrix} 91 \\ 3.288 \\ 2.408 \\ 6.355 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWR} = \min\left(\left(0.75 \cdot N_{sa.7.4} \quad 0.75 \cdot N_{cbg.7.4} \quad 0.7 \cdot N_{pn.7.4}\right)\right)$ $\phi P_{pl.SWR} = 6.809 \cdot kip$

Plate Summary $\dfrac{Vu_{T.SWR}}{\phi P_{pl.SWR}} = 0.067$ or $\left[\dfrac{Vu_{o.s.SWR}}{0.75 \cdot V_{cbg\_op.7.4}} + \dfrac{Vu_{i.s.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.285 < 1.0$

or $\left[\dfrac{Tu_{o.w.SWR}}{\left(0.75 \cdot V_{cbg\_op.7.4}\right)} + \dfrac{Vu_{i.w.SWR}}{\left(0.75 \cdot V_{cbg\_ip.7.4}\right)}\right] = 0.386 < 1.0$

$Result_{SWR}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWR}$ = "not required" < 1.2

$chk_{6.SWR}$ = "OK"

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Weld Between $Plate_{7.3}$ = "#2" and $Plate_{7.4}$ = "#49" along side wall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.RSW.1} := \frac{\left(\sqrt{Vu_{o.s.SWR}^{2} + Vu_{i.s.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.1} = 0.163\cdot in$$

$$l_{w\_Rqrd.RSW.2} := \frac{\left(\sqrt{Vu_{T.SWR}^{2} + Tu_{o.w.SWR}^{2} + Vu_{i.w.SWR}^{2}}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.RSW.2} = 0.342\cdot in$$

$$l_{w\_Rqrd.RSW.3} := \frac{\min\left(w_{pl.7.4}, w_{pl.7.3}\right)}{2} \qquad l_{w\_Rqrd.RSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.RSW} := \max\left(l_{w\_Rqrd.RSW.1}, l_{w\_Rqrd.RSW.2}, l_{w\_Rqrd.RSW.3}\right) \qquad l_{w\_reqd.RSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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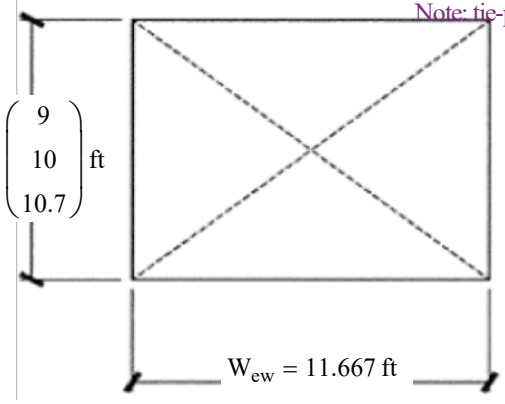
d. Side Wall-To-End Wall



Note: tie-plate weld is located so weld centroid coincides with stud centroid

No. of Plates & maximum plate spacing $No_{pl.EWSW} = 5$

$H_{wall} = \begin{pmatrix} 9 \\ 10 \\ 10.7 \end{pmatrix}$ ft

$$s_{EWSW} := \frac{H_{wall} - 2\cdot(1\,ft + in)}{No_{pl.EWSW} - 1} \qquad s_{SWEW} := s_{EWSW}$$

$s_{EWSW}^{T} = (20.5 \quad 23.5 \quad 25.5)\cdot in$ $s_{SWEW}^{T} = (20.5 \quad 23.5 \quad 25.5)\cdot in$

$A_{t\_r.REW} = 36\,ft^{2}$ Max Trib Triangular Area of Roof-End Wall

$w_{t.r.RSW} = 6$ ft Tributary Width of Roof for Side Wall

$$A_{trib\_EWSW} := H_{wall}\cdot\frac{H_{wall}}{2} \qquad A_{trib\_EWSW} = 147.389\,ft^{2}$$

Max Tributary Triangular Area of end of Sidewall

$$v_{o\_w.EWSW} := v_{w.wall}\cdot A_{trib\_EWSW} \qquad v_{o\_w.EWSW} = 6.383\cdot kip$$

Wind load on end of sidewall

$$v_{o\_s.EWSW} := C_s\cdot\left(WT_{wall\_maxW}\right)\cdot\frac{A_{t\_r.REW}}{W_{ew}\cdot L_{ew}}$$

Seismic load on end of sidewall

$v_{o\_s.EWSW}^{T} = (1.581 \quad 1.36 \quad 1.194 \quad 1.063 \quad 0.959 \quad 0.873 \quad 0.801 \quad 0.74 \quad 0.688)\cdot kip$

$$v_{po\_w.EWSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)\cdot W_{ew}}{2} \qquad v_{po\_w.EWSW} = 2.428\cdot kip$$

Wind load from suction on end wall

$$v_{po\_s.EWSW} := C_s\cdot\left(\omega_{wall}\cdot\max\left(H_{wall}\right)\cdot W_{ew}\right) \qquad v_{po\_s.EWSW} = 1.354\cdot kip$$

Out of Plane Seismic Force on end wall

Overturning Arm at top of wall to top of floor $H_{ot} := \max\left(H_{wall}\right) - t_{floor}$ $H_{ot} = 10$ ft

$$M_{ot\_w.EWSW} := 1.3\left(v_{MWFRS}\cdot\frac{H_{ot}}{2}\cdot\frac{L}{2}\cdot H_{ot} + v_{up}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2}\right)$$

$M_{ot\_w.EWSW}^{T} = (39.9 \quad 43.5 \quad 47.1 \quad 50.7 \quad 54.3 \quad 58 \quad 61.6 \quad 65.2 \quad 68.8)\cdot kip\cdot ft$

$$M_{ot\_s.EWSW} := (C_s)\cdot\left[\left(DL_{r.mx} + 0.25\cdot w_{llr}\right)\cdot W_r\cdot L_r + \frac{\max\left[(\omega_{wall})\cdot\left[\max(H_{wall})\cdot(2L)\right]\right]}{2}\right]\cdot H_{ot}$$

$M_{ot\_s.EWSW}^{T} = (54.7 \quad 58.3 \quad 61.9 \quad 65.5 \quad 69.1 \quad 72.7 \quad 76.3 \quad 79.9 \quad 83.5)\cdot kip\cdot ft$

$$M_{res.EWSW} := DL_{r.mx}\cdot A_{t\_r.REW}\cdot\frac{W_r}{2} + \omega_{wall}\cdot\left(H_{ot}\cdot\frac{L}{2}\right)\cdot W_{ew}$$

$M_{res.EWSW}^{T} = (43.3 \quad 48.6 \quad 53.9 \quad 59.2 \quad 64.5 \quad 69.8 \quad 75.1 \quad 80.4 \quad 85.6)\cdot kip\cdot ft$

$M_{ot\_w.EWSW.net} := M_{ot\_w.EWSW} - 0.9\cdot M_{res.EWSW}$ Net OT Moment, wind

$M_{ot\_w.EWSW.net}^{T} = (0.87 \quad -0.27 \quad -1.42 \quad -2.56 \quad -3.7 \quad -4.85 \quad -5.99 \quad -7.13 \quad -8.28)\cdot kip\cdot ft$

$M_{ot\_s.EWSW.net} := M_{ot\_s.EWSW} - 1.0\cdot M_{res.EWSW}$ Net OT Moment, seismic

$M_{ot\_s.EWSW.net}^{T} = (11.35 \quad 9.65 \quad 7.96 \quad 6.27 \quad 4.58 \quad 2.89 \quad 1.2 \quad -0.49 \quad -2.18)\cdot kip\cdot ft$

$M_{ot.EWSW} := \max(0kip\cdot ft, M_{ot\_s.EWSW.net}, M_{ot\_s.EWSW.net})$ $M_{ot.EWSW} = 11.3\cdot kip\cdot ft$

$v_{ot.EWSW} := \max\left(\frac{M_{ot.EWSW}}{W_{ew}}, 0\cdot kip\right)$ $v_{ot.EWSW} = 0.97\cdot kip$ If OT Moment is > Resisting Moment, decouple to get shear acting on plates.

Design Assumptions: Shear force from roof diaphragm is transferred through wall, so these connections only need to resist vertical in-plane shear created from decoupled OT moment along with vertical uplift of triangular end portion of roof. They also resist direct out-of-plane shear from wind load on traingular-shaped end of sidewall and pullout from wall suction.

$Plate_{7.5}$ = "#2" at inside of end wall $Plate_{7.6}$ = "#2" at end of side wall

$e_{PL.EWSW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.5} := \text{if}\left(s_{L.7.5} = 0in, s_{w.7.5}, \text{if}\left(s_{w.7.5} = 0in, s_{L.7.5}, \min\left(s_{L.7.5}, s_{w.7.5}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.5} := 0.5 + \frac{0.5s_{7.5} + 2e_{PL.EWSW}}{s_{7.5}}$ $F_{7.5} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.5

$s_{7.6} := \text{if}\left(s_{L.7.6} = 0in, s_{w.7.6}, \text{if}\left(s_{w.7.6} = 0in, s_{L.7.6}, \min\left(s_{L.7.6}, s_{w.7.6}\right)\right)\right) = 3\cdot in$ $s_{7.5} = 3\cdot in$

$F_{7.6} := 0.5 + \frac{0.5s_{7.6} + 2e_{PL.EWSW}}{s_{7.6}}$ $F_{7.6} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.6

Factored loads - End Wall - Side wall (End Wall Plates)

$Vu_{i.EWSW} := F_{7.5}\cdot\frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.EWSW} = 0.227\cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Vu_{o.EWSW} = 0.567\cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.EWSW} := F_{7.5}\cdot\frac{1.0\max\left(v_{po\_w.EWSW}, v_{po\_s.EWSW}\right)}{No_{pl.EWSW}}$ $Tu_{o.EWSW} = 0.567\cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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1090 Industrial Blvd.
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Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Factored loads - End Wall - Side wall (Side Wall Plates)

$Vu_{i.SWEW} := F_{7.6} \cdot \frac{v_{ot.EWSW}}{No_{pl.EWSW}}$ $Vu_{i.SWEW} = 0.227 \cdot kip$ 0.9DL + 1.0W in-plane V, overturning

$Vu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Vu_{o.SWEW} = 0.567 \cdot kip$ max(1.0W, 1.0E) out-of-plane V

$Tu_{o.SWEW} := F_{7.6} \cdot \frac{1.0 \max(v_{po\_w.EWSW}, v_{po\_s.EWSW})}{No_{pl.EWSW}}$ $Tu_{o.SWEW} = 0.567 \cdot kip$ max(1.0W, 1.0E) out-of-plane T, wall suction

Plate Calcs Endwall to Sidewall - End Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.5} = 5 \cdot in$ $w_{pl.7.5} = 5 \cdot in$ $x_{Le.7.5} = 1 \cdot in$ $x_{we.7.5} = 2.5 \cdot in$ $t_{pl.7.5} = 0.313 \cdot in$

$h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.5} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.5} = 1.5 \cdot in$

Shear Studs: $n_{stud.7.5} = 2$ $d_{0.7.5} = 0.5 \cdot in$ $A_{SE.7.5} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $N_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.5} = 1.719 \cdot in$ $c_{a1\_po.7.5} = 10.5 \cdot in$ $c_{a2\_po.7.5} = 4 \cdot in$ $s1_{7.5} = 3 \cdot in$ $s2_{7.5} = 0 \cdot in$

Design Factors: $k_{c.7.5} = 24$ $A_{Nc.7.5} = 42.07 \cdot in^2$ $A_{Nco.7.5} = 26.59 \cdot in^2$ $e'_{N.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.5} = 1.00$ $\psi_{ed\_N.7.5} = 1.00$ $\psi_{c\_N.7.5} = 1.00$ $\psi_{cp\_N.7.5} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.5} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.5} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.5} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.5} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.5} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.5} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.5} = n_{stud.7.5} \cdot A_{SE.7.5} \cdot f_{uta}$ $V_{sa.7.5} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.5} = 1.719 \cdot in$ $h_{min.7.5} = 4 \cdot in$ $c_{a1\_op.7.5} = 4 \cdot in$ $c_{a2\_op.7.5} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.5} = 60 \cdot in^2$ $A_{Vco\_op.7.5} = 72 \cdot in^2$ $e'_{V.7.5} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.5} = 1.00$ $\psi_{ed\_Vop.7.5} = 1.00$ $\psi_{c\_Vop.7.5} = 1.20$ cracking, edge reinf provided

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Basic Breakout Strength $V_{b\_op.7.5} = 3.89 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.5} = 3.89 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.5a} = 0 \cdot in$ $s2_{7.5a} = 3 \cdot in$ $l_{e.7.5} = 1.719 \cdot in$ $h_{ef.7.5} = 1.719 \cdot in$ $h_{a.7.5} = 4 \cdot in$ $h_{min.7.5a} = 4 \cdot in$

$c_{a1\_ip.7.5} = 13.5 \cdot in$ $c_{a2\_ip.7.5} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.5} = 97 \cdot in^2$ $A_{Vco\_ip.7.5} = 820 \cdot in^2$ $e'_{V.7.5a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.5} = 1.00$ $\psi_{ed\_Vip.7.5} = 0.76$ $\psi_{c\_Vip.7.5} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.5} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.5} = 2.6 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.5} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.5} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.5} = k_{cp.7.5} \cdot N_{cbg.7.5}$ $V_{cpg.7.5} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.5} \\ 0.75 \cdot V_{cbg\_op.7.5} \\ 0.75 \cdot V_{cbg\_ip.7.5} \\ 0.7V_{cpg.7.5} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 2.915 \\ 1.947 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EWSW} = \min\left(\left(0.75 \cdot N_{sa.7.5} \quad 0.75 \cdot N_{cbg.7.5} \quad 0.7 \cdot N_{pn.7.5}\right)\right)$ $\phi P_{pl.EWSW} = 4.305 \cdot kip$

Plate Summary $\dfrac{Tu_{o.EWSW}}{\phi P_{pl.EWSW}} = 0.132$ or $\left[\dfrac{Vu_{o.EWSW}}{0.75 \cdot V_{cbg\_op.7.5}} + \dfrac{Vu_{i.EWSW}}{\left(0.75 \cdot V_{cbg\_ip.7.5}\right)}\right] = 0.311$ < 1.0

$Result_{EWSW} =$ "< 0.2, full strength shear/tension permitted" $IC_{pl.EWSW} =$ "not required" < 1.2

$chk_{6.EWSW} =$ "OK"

Plate Calcs Endwall to Sidewall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.6} = 5 \cdot in$ $w_{pl.7.6} = 5 \cdot in$ $x_{Le.7.6} = 1 \cdot in$ $x_{we.7.6} = 2.5 \cdot in$ $t_{pl.7.6} = 0.313 \cdot in$

$h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.6} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.6} = 0 \cdot in$

Shear Studs: $n_{stud.7.6} = 2$ $d_{0.7.6} = 0.5 \cdot in$ $A_{SE.7.6} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $N_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.6} = 1.719 \cdot in$ $c_{a1\_po.7.6} = 10.5 \cdot in$ $c_{a2\_po.7.6} = 2.5 \cdot in$ $s1_{7.6} = 3 \cdot in$ $s2_{7.6} = 0 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Design Factors: $k_{c.7.6} = 24$ $A_{Nc.7.6} = 41.66 \cdot in^2$ $A_{Nco.7.6} = 26.59 \cdot in^2$ $e'_{N.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.6} = 1.00$ $\psi_{ed\_N.7.6} = 0.99$ $\psi_{c\_N.7.6} = 1.00$ $\psi_{cp\_N.7.6} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.6} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.6} = 5.63 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.6} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.6} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.6} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.6} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.6} = n_{stud.7.6} \cdot A_{SE.7.6} \cdot f_{uta}$ $V_{sa.7.6} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.6} = 1.719 \cdot in$ $h_{min.7.6} = 2.5 \cdot in$ $c_{a1\_op.7.6} = 2.5 \cdot in$ $c_{a2\_op.7.6} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.6} = 26 \cdot in^2$ $A_{Vco\_op.7.6} = 28 \cdot in^2$ $e'_{V.7.6} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.6} = 1.00$ $\psi_{ed\_Vop.7.6} = 1.00$ $\psi_{c\_Vop.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.6} = 1.92 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.6} = 2.15 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.6a} = 0 \cdot in$ $s2_{7.6a} = 3 \cdot in$ $l_{e.7.6} = 1.719 \cdot in$ $h_{ef.7.6} = 1.719 \cdot in$ $h_{a.7.6} = 4 \cdot in$ $h_{min.7.6a} = 4 \cdot in$ $c_{a1\_ip.7.6} = 13.5 \cdot in$ $c_{a2\_ip.7.6} = 2.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.6} = 91 \cdot in^2$ $A_{Vco\_ip.7.6} = 820 \cdot in^2$ $e'_{V.7.6a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.6} = 1.00$ $\psi_{ed\_Vip.7.6} = 0.74$ $\psi_{c\_Vip.7.6} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.6} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.6} = 2.36 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.6} = 1.00$ *The pryout resistance is approximated by 1-2 times the anchor tensile resistance.*

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.6} = 5.63 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.6} = k_{cp.7.6} \cdot N_{cbg.7.6}$ $V_{cpg.7.6} = 5.63 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.6} \\ 0.75 \cdot V_{cbg\_op.7.6} \\ 0.75 \cdot V_{cbg\_ip.7.6} \\ 0.7V_{cpg.7.6} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 1.613 \\ 1.774 \\ 3.943 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Tensile Allowable Summary

$\phi P_{pl.SWEW} = \min\left(\left(0.75 \cdot N_{sa.7.6} \quad 0.75 \cdot N_{cbg.7.6} \quad 0.7 \cdot N_{pn.7.6}\right)\right)$ $\phi P_{pl.SWEW} = 4.224 \cdot kip$

Plate Summary $\dfrac{Tu_{o.SWEW}}{\phi P_{pl.SWEW}} = 0.134$ or $\left[\dfrac{Vu_{o.SWEW}}{0.75 \cdot V_{cbg\_op.7.6}} + \dfrac{Vu_{i.SWEW}}{\left(0.75 \cdot V_{cbg\_ip.7.6}\right)}\right] = 0.479$ < 1.0

$Result_{SWEW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWEW}$ = "not required" < 1.2

$chk_{6.SWEW}$ = "OK"

Weld Between $Plate_{7.5}$ = "#2" and $Plate_{7.6}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.EWSW.1} := \frac{\left(\sqrt{Tu_{o.EWSW}^2 + Vu_{o.EWSW}^2 + Vu_{i.EWSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.1} = 0.244 \cdot in$$

$$l_{w\_Rqrd.EWSW.2} := \frac{\left(\sqrt{Tu_{o.SWEW}^2 + Vu_{o.SWEW}^2 + Vu_{i.SWEW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in} \qquad l_{w\_Rqrd.EWSW.2} = 0.244 \cdot in$$

$$l_{w\_Rqrd.EWSW.3} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5 \cdot in$$

$l_{w\_reqd.EWSW} := \max\left(l_{w\_Rqrd.EWSW.1}, l_{w\_Rqrd.EWSW.2}, l_{w\_Rqrd.EWSW.3}\right)$ $l_{w\_reqd.EWSW} = 2.5 \cdot in$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

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e. Floor Panel-To-End Wall

$A_{t\_r.FEW} := A_{t\_r.REW}$ $A_{t\_r.FEW} = 36\,ft^2$ Max Tributary Triangular Area of Roof for End Wall

$v_{DL.FEW} := DL_{r.mx} \cdot \frac{A_{t\_r.FEW}}{W_{ew}} + \omega_{wall} \cdot \max\left(H_{wall}\right)$ $v_{DL.FEW} = 0.649 \cdot klf$ Weight of roof and endwall

$v_{LL.FEW} := w_{llr} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{LL.FEW} = 0.108 \cdot klf$ Roof LL

$t_{o.s.FEW} := C_s\left(\omega_{wall} \cdot \frac{\max\left(H_{wall}\right)}{4}\right)$ $t_{o.s.FEW} = 0.029 \cdot klf$ out-of-plane T, seismic

$v_{i.s.FEW} := v_{NS\_smaxW}$ $v_{i.s.FEW} = 0.358 \cdot klf$ in-of-plane V, seismic

$v_{i.w.FEW} := v_{NS\_wmaxW}$ $v_{i.w.FEW} = 0.372 \cdot klf$ in-of-plane V, wind

$v_{up.w.FEW} := v_{up} \cdot \frac{A_{t\_r.FEW}}{W_{ew}}$ $v_{up.w.FEW} = 0.2 \cdot klf$ Uplift wind load from roof

$t_{p.w.FEW} := v_{w.side} \cdot \frac{W_{ew}}{4}$ $t_{p.w.FEW} = 0.114 \cdot klf$ Wind load suction on each of four sides of end wall

$H_{ot} = 10\,ft$ OT Arm at top of wall to top of floor

Loads for consideration of Overturning

$$M_{ot.s.FEW} := 1.0\left[\left(C_s\right) \cdot \left[\left(DL_{r.mx} + 0.25 \cdot w_{llr}\right) \cdot W_r \cdot L_r + \frac{\left(\omega_{wall}\right) \cdot \left[\max\left(H_{wall}\right) \cdot (2L)\right]}{2}\right]\right] \cdot H_{ot}$$ OT Moment, seismic

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$M_{ot.s.FEW}^{T} = (36.1 \quad 42 \quad 47.9 \quad 53.9 \quad 59.8 \quad 65.7 \quad 71.6 \quad 77.5 \quad 83.5)\cdot kip\cdot ft$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No.
of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of
Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$M_{ot.w.FEW} := 1.3\cdot\left[v_{MWFRS}\cdot W_{ew}\cdot H_{ot}\cdot\frac{H_{ot}}{2} + v_{up}\cdot W_{ew}\cdot\begin{bmatrix} 0.5\cdot(L_0)^2 \\ 0.5\cdot(L_1)^2 \\ 0.5\cdot(L_2)^2 \\ 0.5\cdot(L_3)^2 \\ 0.5\cdot(L_4)^2 \\ 0.5\cdot(L_5)^2 \\ 0.5\cdot(L_6)^2 \\ 0.5\cdot(L_7)^2 \\ 0.5\cdot(L_8)^2 \end{bmatrix}\right]$$

OT Moment, wind

$M_{ot.w.FEW}^{T} = (112.9 \quad 138.4 \quad 167.8 \quad 201.2 \quad 238.5 \quad 279.7 \quad 324.8 \quad 373.9 \quad 426.9)\cdot kip\cdot ft$

$M_{res.FEW} := WT_{r\_minW}\cdot\frac{L}{2} + WT_{wall\_minW}\cdot\frac{L}{2}$

OT Moment, resisting

$M_{res.FEW}^{T} = (250.2 \quad 291.9 \quad 333.6 \quad 375.3 \quad 417 \quad 458.7 \quad 500.4 \quad 542.1 \quad 583.9)\cdot kip\cdot ft$

$M_{ot.w.FEW.net} := M_{ot.w.FEW} - 0.9\cdot M_{res.FEW}$

Net OT Moment, wind

$M_{ot.w.FEW.net}^{T} = (-112.3 \quad -124.4 \quad -132.5 \quad -136.6 \quad -136.9 \quad -133.2 \quad -125.6 \quad -114 \quad -98.6)\cdot kip\cdot ft$

$M_{ot.s.FEW.net} := M_{ot.s.FEW} - 1.0\cdot M_{res.FEW}$

Net OT Moment, seismic

$M_{ot.s.FEW.net}^{T} = (-214.1 \quad -249.9 \quad -285.7 \quad -321.5 \quad -357.3 \quad -393 \quad -428.8 \quad -464.6 \quad -500.4)\cdot kip\cdot ft$

$M_{ot.FEW} := \max(0kip\cdot ft, M_{ot.w.FEW.net}, M_{ot.s.FEW.net})$ $\quad M_{ot.FEW} = 0\cdot kip\cdot ft$

Net OT Moment, max(wind, seismic)

$v_{ot.w.FEW} := \max\left[\frac{M_{ot.w.FEW} - M_{res.FEW}}{2(L)}, 0\cdot kip\right]$ $\quad v_{ot.w.FEW} = 0\cdot kip$

$v_{ot.s.FEW} := \max\left(\frac{M_{ot.s.FEW} - M_{res.FEW}}{2L}, 0\cdot kip\right)$ $\quad v_{ot.s.FEW} = 0\cdot kip$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.7}$ = "#1B" at top of floor

$Plate_{7.8}$ = "#2" at inside face of end wall

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

No. of Plates & maximum plate spacing $No_{pl.FEW} = 7$

$$s_{F\_EW} := \frac{W_{ew} - 2(18in + 4.25in)}{No_{pl.FEW} - 1} \quad s_{F\_EW} = 15.92 \cdot in$$

$$s_{EW\_F} := \frac{W_{ew} - 2(18in)}{No_{pl.FEW} - 1} \quad s_{EW\_F} = 17.33 \cdot in$$

$e_{PL.F\_EW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.7} := if\left(s_{L.7.7} = 0in, s_{w.7.7}, if\left(s_{w.7.7} = 0in, s_{L.7.7}, min\left(s_{L.7.7}, s_{w.7.7}\right)\right)\right) = 3 \cdot in$ $s_{7.7} = 3 \cdot in$

$$F_{7.7} := 0.5 + \frac{0.5s_{7.7} + 2e_{PL.F\_EW}}{s_{7.7}} \quad F_{7.7} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.7

$s_{7.8} := if\left(s_{L.7.8} = 0in, s_{w.7.8}, if\left(s_{w.7.8} = 0in, s_{L.7.8}, min\left(s_{L.7.8}, s_{w.7.8}\right)\right)\right) = 3 \cdot in$ $s_{7.7} = 3 \cdot in$

$$F_{7.8} := 0.5 + \frac{0.5s_{7.8} + 2e_{PL.F\_EW}}{s_{7.8}} \quad F_{7.8} = 1.167$$ Force Magnification Factor for Eccentricity, Plate 7.8

Factored loads - Floor - Endwall (Floor Plates)

$$Vu_{i.s.FEW} := F_{7.7} \cdot \left(1.0v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{\max(H_{wall})}{4} \cdot W_{ew}\right) \quad Vu_{i.s.FEW} = 1.118 \cdot kip$$ 1.0E in-plane V, seismic roof diaphragm

$$Vu_{i.w.FEW} := F_{7.7} \cdot 1.0v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} \quad Vu_{i.w.FEW} = 0.724 \cdot kip$$ 1.0W in-plane V, wind roof diaphragm

$Tu_{po.s1.FEW} := F_{7.7} \cdot 0.3C_s \cdot \left(t_{o.s.FEW} \cdot W_{ew}\right)$ $Tu_{po.s1.FEW} = 0.03 \cdot kip$ 0.3E pullout, seismic

$$Tu_{po.w.FEW} := F_{7.7} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} \quad Tu_{po.w.FEW} = 0.443 \cdot kip$$ 1.0W pullout, wind bot of wall & bot half of 2 sides of wall

$$Vu_{o.9dl.FEW} := F_{7.7} \cdot \left(0.9v_{DL.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} \quad Vu_{o.9dl.FEW} = 1.136 \cdot kip$$ 0.9D Shear from DL only

$$V_{ot.w.FEW} := F_{7.7} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}} \quad V_{ot.w.FEW} = 0 \cdot kip$$

1.0W
Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$$V_{ot.s.FEW} := F_{7.7} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}} \quad V_{ot.s.FEW} = 0 \cdot kip$$

1.0E

$$Vu_{o.wup.FEW} := F_{7.7} \cdot \max\left[\left(1.0v_{up.w.FEW}\right) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip\right]$$

$Vu_{o.wup.FEW} = 0 \cdot kip$ 0.9D+1.0W

$Vu_{o.sup.FEW} := F_{7.7} \cdot \max\left(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip\right)$

Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.sup.FEW} = 0 \cdot kip$ 0.9D+1.0E

$$Vu_{o.12D.16L.FEW} := F_{7.7} \cdot \left[1.2\left(v_{DL.FEW}\right) + 1.6v_{LL.FEW}\right] \cdot \frac{W_{ew}}{No_{pl.FEW}}$$

$Vu_{o.12D.16L.FEW} = 1.851 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

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Factored loads - Floor - Endwall (Floor Wall Plates)

$Tu_{po.w.EWF} := F_{7.8} \cdot 1.0 \cdot 2 \cdot t_{p.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Tu_{po.w.EWF} = 0.443 \cdot kip$ pullout, wind suction at bot of wall and bot half of two sides of wall

$Tu_{po.s2.EWF} := F_{7.8} \cdot 0.3 C_s \cdot (t_{o.s.FEW} \cdot W_{ew})$ $Tu_{po.s2.EWF} = 0.03 \cdot kip$ 0.3E pullout, seismic

$Vu_{i.s.EWF} := F_{7.8} \cdot \left(1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}} + C_s \omega_{wall} \cdot \frac{\max(H_{wall})}{4} \cdot W_{ew} \right)$

$Vu_{i.s.EWF} = 1.118 \cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w2.EWF} := F_{7.8} \cdot 1.0 v_{i.w.FEW} \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{i.w2.EWF} = 0.724 \cdot kip$ 1.0W in-plane V, wind roof diaphragm

$Vu_{o.9dl.EWF} := F_{7.8} \cdot (0.9 v_{DL.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}}$ $Vu_{o.9dl.EWF} = 1.136 \cdot kip$ 0.9D Shear from DL only

$V_{ot.w.EWF} := F_{7.8} \cdot \frac{v_{ot.w.FEW}}{No_{pl.FEW}}$ $V_{ot.w.EWF} = 0 \cdot kip$ 1.0W

$V_{ot.s.EWF} := F_{7.8} \cdot \frac{v_{ot.s.FEW}}{No_{pl.FEW}}$ $V_{ot.s.EWF} = 0 \cdot kip$ 1.0E

Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$Vu_{o.wdn.EWF} := F_{7.8} \cdot \left[Vu_{o.9dl.FEW} - (1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} + V_{ot.w.FEW} \right]$

$Vu_{o.wdn.EWF} = 0.937 \cdot kip$ 0.9D+1.0W Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.wup.EWF} := F_{7.8} \cdot \max\left[(1.0 v_{up.w.FEW}) \cdot \frac{W_{ew}}{No_{pl.FEW}} - Vu_{o.9dl.FEW} + V_{ot.w.FEW}, 0 \cdot kip \right]$

$Vu_{o.wup.EWF} = 0 \cdot kip$

$Vu_{o.sdn.EWF} := F_{7.8} \cdot (Vu_{o.9dl.FEW} + V_{ot.s.FEW})$ $Vu_{o.sdn.EWF} = 1.326 \cdot kip$ 0.9D+1.0E Out-of-plane V, gravity &/or roof uplift & decoupled OT moment.

$Vu_{o.sup.EWF} := F_{7.8} \cdot \max(V_{ot.s.FEW} - Vu_{o.9dl.FEW}, 0 \cdot kip)$ $Vu_{o.sup.EWF} = 0 \cdot kip$

$Vu_{o.12D.16L.EWF} := F_{7.8} \cdot [1.2(v_{DL.FEW}) + 1.6 v_{LL.FEW}] \cdot \frac{W_{ew}}{No_{pl.FEW}}$

$Vu_{o.12D.16L.EWF} = 1.851 \cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Plate Calcs Floor to Endwall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.7} = 5 \cdot in$ $w_{pl.7.7} = 5 \cdot in$ $x_{Le.7.7} = 1 \cdot in$ $x_{we.7.7} = 4 \cdot in$ $t_{pl.7.7} = 0.313 \cdot in$

$h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.7} = 16 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.7} = 0 \cdot in$

Shear Studs: $n_{stud.7.7} = 2$ $d_{0.7.7} = 0.5 \cdot in$ $A_{SE.7.7} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $N_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.7} = 3.719 \cdot in$ $c_{a1\_po.7.7} = 14.5 \cdot in$ $c_{a2\_po.7.7} = 4 \cdot in$ $s1_{7.7} = 3 \cdot in$ $s2_{7.7} = 0 \cdot in$

Design Factors: $k_{c.7.7} = 24$ $A_{Nc.7.7} = 140.34 \cdot in^2$ $A_{Nco.7.7} = 124.48 \cdot in^2$ $e'_{N.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.7} = 1.00$ $\psi_{ed\_N.7.7} = 0.92$ $\psi_{c\_N.7.7} = 1.00$ $\psi_{cp\_N.7.7} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.7} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.7} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.7} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.7} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.7} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.7} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.7} = n_{stud.7.7} \cdot A_{SE.7.7} \cdot f_{uta}$ $V_{sa.7.7} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.7} = 3.719 \cdot in$ $h_{min.7.7} = 4 \cdot in$ $c_{a1\_op.7.7} = 4 \cdot in$ $c_{a2\_op.7.7} = 14.5 \cdot in$

Design Factors: $A_{Vc\_op.7.7} = 60 \cdot in^2$ $A_{Vco\_op.7.7} = 72 \cdot in^2$ $e'_{V.7.7} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.7} = 1.00$ $\psi_{ed\_Vop.7.7} = 1.00$ $\psi_{c\_Vop.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.7} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.7} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.7a} = 0 \cdot in$ $s2_{7.7a} = 3 \cdot in$ $l_{e.7.7} = 3.719 \cdot in$ $h_{ef.7.7} = 3.719 \cdot in$ $h_{a.7.7} = 8 \cdot in$ $h_{min.7.7a} = 8 \cdot in$
$c_{a1\_ip.7.7} = 17.5 \cdot in$ $c_{a2\_ip.7.7} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.7} = 242 \cdot in^2$ $A_{Vco\_ip.7.7} = 1378 \cdot in^2$ $e'_{V.7.7a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.7} = 1.00$ $\psi_{ed\_Vip.7.7} = 0.75$ $\psi_{c\_Vip.7.7} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.7} = 41.5 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.7} = 6.52 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.7} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.7} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.7} = k_{cp.7.7} \cdot N_{cbg.7.7}$ $V_{cpg.7.7} = 23.83 \cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.7} \\ 0.75\cdot V_{cbg\_op.7.7} \\ 0.75\cdot V_{cbg\_ip.7.7} \\ 0.7V_{cpg.7.7} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.891 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$$\phi P_{pl.F\_EW} = \min\left(\left(0.75\cdot N_{sa.7.7} \quad 0.75\cdot N_{cbg.7.7} \quad 0.7\cdot N_{pn.7.7}\right)\right) \qquad \phi P_{pl.F\_EW} = 8.935\cdot kip$$

Plate Summary

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.05 \quad \text{or} \quad \left[\frac{Vu_{o.wup.FEW}}{0.75\cdot V_{cbg\_op.7.7}} + \frac{Vu_{i.w.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.148 < 1.0$$

$$\frac{Tu_{po.w.FEW}}{\phi P_{pl.F\_EW}} = 0.05 \quad \text{or} \quad \left[\frac{Vu_{o.sup.FEW}}{\left(0.75\cdot V_{cbg\_op.7.7}\right)} + \frac{Vu_{i.s.FEW}}{\left(0.75\cdot V_{cbg\_ip.7.7}\right)}\right] = 0.229 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.FEW}}{0.75\cdot V_{cbg\_op.7.7}}\right) = 0.544 < 1.0$$

$Result_{F\_EW}$ = "< 0.2, full strength shear/tension permitted"

$IC_{pl.F\_EW}$ = "not required" < 1.2

$chk_{6.F\_EW}$ = "OK"

Plate Calcs Floor to Endwall - Endwall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.8} = 5\cdot in$ $w_{pl.7.8} = 5\cdot in$ $x_{Le.7.8} = 1\cdot in$ $x_{we.7.8} = 2.5\cdot in$ $t_{pl.7.8} = 0.313\cdot in$

$h_{ef.7.8} = 1.719\cdot in$ $h_{a.7.8} = 4\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate on end of sidewall panel: $x_{es\_s.7.8} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate on end of sidewall panel: $x_{es\_e.7.8} = 4\cdot in$

Shear Studs: $n_{stud.7.8} = 2$ $d_{0.7.8} = 0.5\cdot in$ $A_{SE.7.8} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.8} = n_{stud.7.8}\cdot A_{SE.7.8}\cdot f_{uta}$ $N_{sa.7.8} = 70\cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.8} = 1.719\cdot in$ $c_{a1\_po.7.8} = 10.5\cdot in$ $c_{a2\_po.7.8} = 6.5\cdot in$ $s1_{7.8} = 3\cdot in$ $s2_{7.8} = 0\cdot in$

Design Factors: $k_{c.7.8} = 24$ $A_{Nc.7.8} = 42.07\cdot in^2$ $A_{Nco.7.8} = 26.59\cdot in^2$ $e'_{N.7.8} = 0\cdot in$

Adjustment Factors: $\psi_{ec\_N.7.8} = 1.00$ $\psi_{ed\_N.7.8} = 1.00$ $\psi_{c\_N.7.8} = 1.00$ $\psi_{cp\_N.7.8} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.8} = 3.63\cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.8} = 5.74\cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.8} = 0.785\cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.8} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.8} = 28.27\cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.8} = 56.55\cdot kip$

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.8} = n_{stud.7.8} \cdot A_{SE.7.8} \cdot f_{uta}$ $V_{sa.7.8} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.8} = 1.719 \cdot in$ $h_{min.7.8} = 4 \cdot in$ $c_{a1\_op.7.8} = 6.5 \cdot in$ $c_{a2\_op.7.8} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.8} = 90 \cdot in^2$ $A_{Vco\_op.7.8} = 190 \cdot in^2$ $e'_{V.7.8} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.8} = 1.00$ $\psi_{ed\_Vop.7.8} = 1.00$ $\psi_{c\_Vop.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.8} = 8.05 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.8} = 4.57 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.8a} = 0 \cdot in$ $s2_{7.8a} = 3 \cdot in$ $l_{e.7.8} = 1.719 \cdot in$ $h_{ef.7.8} = 1.719 \cdot in$ $h_{a.7.8} = 4 \cdot in$ $h_{min.7.8a} = 4 \cdot in$

$c_{a1\_ip.7.8} = 13.5 \cdot in$ $c_{a2\_ip.7.8} = 6.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.8} = 107 \cdot in^2$ $A_{Vco\_ip.7.8} = 820 \cdot in^2$ $e'_{V.7.8a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.8} = 1.00$ $\psi_{ed\_Vip.7.8} = 0.80$ $\psi_{c\_Vip.7.8} = 1.20$ cracking may occur, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.8} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.8} = 3 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.8} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.8} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.8} = k_{cp.7.8} \cdot N_{cbg.7.8}$ $V_{cpg.7.8} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.8} \\ 0.75 \cdot V_{cbg\_op.7.8} \\ 0.75 \cdot V_{cbg\_ip.7.8} \\ 0.7V_{cpg.7.8} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.43 \\ 2.253 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.EW\_F} = \min\left(\left(0.75 \cdot N_{sa.7.8} \quad 0.75 \cdot N_{cbg.7.8} \quad 0.7 \cdot N_{pn.7.8}\right)\right)$ $\phi P_{pl.EW\_F} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{po.s2.EWF}}{\phi P_{pl.EW\_F}} = 0.007 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.s.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.883 \quad < 1.0$$

$$\frac{Tu_{po.w.EWF}}{\phi P_{pl.EW\_F}} = 0.103 \quad \text{or} \quad \max\left[\frac{\max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)}{\left(0.75 \cdot V_{cbg\_op.7.8}\right)} + \frac{Vu_{i.w2.EWF}}{\left(0.75 \cdot V_{cbg\_ip.7.8}\right)}\right] = 0.594 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.EWF}}{0.75 \cdot V_{cbg\_op.7.8}}\right) = 0.54 \quad < 1.0$$

$Result_{EW\_F}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.EW\_F}$ = "not required" < 1.2

$chk_{6.EW\_F}$ = "OK"

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:
APPROVED BY NTA INC.
Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Weld Between $Plate_{7.7}$ = "#1B" and $Plate_{7.8}$ = "#2" along endwall

Will utilize a rod at the intersection of the plates and weld the panels together

$$l_{w\_Rqrd.FEW.1} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + Vu_{o.wup.FEW}^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.1} = 0.249\cdot in$$

$$l_{w\_Rqrd.FEW.2} := \frac{\left(\sqrt{Tu_{po.s1.FEW}^2 + Vu_{o.sup.FEW}^2 + Vu_{i.s.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.2} = 0.328\cdot in$$

$$l_{w\_Rqrd.FEW.3} := \frac{\left(\sqrt{Tu_{po.w.FEW}^2 + \max\left(Vu_{o.wdn.EWF}, Vu_{o.wup.EWF}\right)^2 + Vu_{i.w.FEW}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.3} = 0.37\cdot in$$

$$l_{w\_Rqrd.FEW.4} := \frac{\left(\sqrt{Tu_{po.s2.EWF}^2 + \max\left(Vu_{o.sdn.EWF}, Vu_{o.sup.EWF}\right)^2 + Vu_{i.s.EWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.508\cdot in$$

$$l_{w\_Rqrd.FEW.5} := \frac{\max\left(Vu_{o.12D.16L.FEW}, Vu_{o.12D.16L.EWF}\right)}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FEW.4} = 0.508\cdot in$$

$$l_{w\_Rqrd.FEW.6} := \frac{\min\left(w_{pl.7.6}, w_{pl.7.5}\right)}{2} \qquad l_{w\_Rqrd.EWSW.3} = 2.5\cdot in$$

$$l_{w\_reqd.FEW} := \max\left(l_{w\_Rqrd.FEW.1}, l_{w\_Rqrd.FEW.2}, l_{w\_Rqrd.FEW.3}, l_{w\_Rqrd.FEW.4}, l_{w\_Rqrd.FEW.5}, l_{w\_Rqrd.FEW.6}\right)$$

$$l_{w\_reqd.EWSW} = 2.5\cdot in$$

minimum weld length required considering an effective throat of 1/8"

Will provide minimum of 3" at each plate

****

****

f. Floor Panel-To-Side Wall

$w_{t.r.FSW} := w_{t.r.RSW}$ $w_{t.r.FSW} = 6\ ft$ Tributary Width of Roof for Side Wall

$v_{dl.FSW} := DL_{r.mx}\cdot w_{t.r.FSW} + \omega_{wall}\cdot\max\left(H_{wall}\right)$ $v_{dl.FSW} = 0.806\cdot klf$ Downward load from weight of roof and sidewall

$v_{ll.FSW} := w_{llr}\cdot w_{t.r.FSW}$ $v_{ll.FSW} = 0.21\cdot klf$ Downward load from roof LL

$t_{o.s.FSW} := C_s\left(\omega_{wall}\cdot\frac{\max\left(H_{wall}\right)}{2}\right)$ $t_{o.s.FSW} = 0.058\cdot klf$ Seismic load in NS direction along bottom of side wall

$v_{i.s.FSW} := v_{EW\_smaxW}$ $v_{i.s.FSW} = 0.348\cdot klf$ Shear in shear wall due to direct seismic load in EW direction

$v_{i.w.SW} := v_{EW\_wmaxW}$ $v_{i.w.SW} = 0.0323\cdot klf$ Shear in shear wall due to direct wind load in EW direction

$v_{up.w.FSW} := v_{up}\cdot w_{t.r.FSW}$ $v_{up.w.FSW} = 0.388\cdot klf$ Uplift wind load from roof

$t_{po.w.FSW} := v_{w.side}\cdot\frac{\max\left(H_{wall}\right)}{2}$ $t_{po.w.FSW} = 0.208\cdot klf$ Wind load suction on bottom of side wall

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Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Loads for consideration of Overturning

$M_{ot.w.FSW} := M_{ot.w.FEW}$ OT Moment, wind

$M_{ot.w.FSW}^T = (112.9 \quad 138.4 \quad 167.8 \quad 201.2 \quad 238.5 \quad 279.7 \quad 324.8 \quad 373.9 \quad 426.9)\cdot kip\cdot ft$

$M_{ot.s.FSW} := M_{ot.s.FEW}$ OT Moment, seismic

$M_{ot.s.FSW}^T = (36.1 \quad 42 \quad 47.9 \quad 53.9 \quad 59.8 \quad 65.7 \quad 71.6 \quad 77.5 \quad 83.5)\cdot kip\cdot ft$

$M_{ot.FSW} := M_{ot.FEW}$ $M_{ot.FSW} = 0\cdot kip\cdot ft$ OT Moment, max(wind, seismic)

$M_{res.FSW} := M_{res.FEW}$ OT Moment, resisting

$M_{res.FSW}^T = (250.2 \quad 291.9 \quad 333.6 \quad 375.3 \quad 417 \quad 458.7 \quad 500.4 \quad 542.1 \quad 583.9)\cdot kip\cdot ft$

$v_{ot.w.FSW} := \max\left(\frac{M_{ot.w.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right)$ $v_{ot.w.FSW} = 0\cdot kip$

Decoupled OT Moment. Load is divided in half, shared by both end wall & side wall

$v_{ot.s.FSW} := \max\left(\frac{M_{ot.s.FSW} - 0.9M_{res.FSW}}{2\min(W_{Bldg}, L)}, 0\cdot kip\right)$ $v_{ot.s.FSW} = 0\cdot kip$

Design Assumption: Sidewall-to-endwall connectors transfer load, thisus engages sidewall to resist decoupled moment.

$Plate_{7.9}$ = "#1B" at top of floor $Plate_{7.10}$ = "#2" at inside face of side wall

Maximum plate spacing $s_{max.FSW} = 24\cdot in$ Minimum # of Plates = 4

$e_{PL.F\_SW} := 0.25in$ Allowance for plate mis-alignment

$s_{7.9} := if\left(s_{L.7.9} = 0in, s_{w.7.9}, if\left(s_{w.7.9} = 0in, s_{L.7.9}, \min\left(s_{L.7.9}, s_{w.7.9}\right)\right)\right) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.9} := 0.5 + \frac{0.5s_{7.9} + 2e_{PL.F\_SW}}{s_{7.9}}$ $F_{7.9} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.9

$s_{7.10} := if\left(s_{L.7.10} = 0in, s_{w.7.10}, if\left(s_{w.7.10} = 0in, s_{L.7.10}, \min\left(s_{L.7.10}, s_{w.7.10}\right)\right)\right) = 3\cdot in$ $s_{7.9} = 3\cdot in$

$F_{7.10} := 0.5 + \frac{0.5s_{7.10} + 2e_{PL.F\_SW}}{s_{7.10}}$ $F_{7.10} = 1.167$ Force Magnification Factor for Eccentricity, Plate 7.10

Factored loads - Floor - Side Wall (Floor Plates)

$Tu_{o.w.FSW} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$	$Tu_{o.w.FSW} = 0.486\cdot kip$	1.0W	T, wall suction
$Tu_{o.s.FSW} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$	$Tu_{o.s.FSW} = 0.135\cdot kip$	0.3W	T, pullout seismic
$Vu_{i.s.FSW} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$	$Vu_{i.s.FSW} = 0.243\cdot kip$	1.0E	in-plane V, seismic roof diaphragm
$Vu_{i.w.FSW} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$	$Vu_{i.w.FSW} = 0.075\cdot kip$	1.0W	in-plane V roof diaphragm
$Vu_{o\_09dl.FSW} := (0.9v_{dl.FSW})\cdot s_{max.FSW}$	$Vu_{o\_09dl.FSW} = 1.45\cdot kip$	0.9D	Shear from DL only

$V_{ot\_w.FSW} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.FSW} = 0\cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.FSW} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.FSW} = 0\cdot kip$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$Vu_{o.w.FSW} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.FSW} \qquad +-V_{ot\_w.FSW}$$

$Vu_{o.w.FSW} = 2.598\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.FSW} := F_{7.10}\cdot\max\left(V_{ot\_s.FSW} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.FSW} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$

$Vu_{o.s.FSW} = 1.692\cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.FSW} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$

$Vu_{o.12D.16L.FSW} = 3.04\cdot kip$ 1.2D+1.6L Out-of-Plane V, DL + LL

Factored loads - Floor - Side Wall (Side Wall Plates)

$Tu_{o.w.SWF} := F_{7.10}\cdot t_{po.w.FSW}\cdot s_{max.FSW}$ $Tu_{o.w.SWF} = 0.486\cdot kip$ 1.0W T, wall suction

$Tu_{o.s.SWF} := F_{7.10}\cdot t_{o.s.FSW}\cdot s_{max.FSW}$ $Tu_{o.s.SWF} = 0.135\cdot kip$ 0.3W T, pullout seismic

$Vu_{i.s.SWF} := F_{7.10}\cdot 0.3v_{i.s.FSW}\cdot s_{max.FSW}$ $Vu_{i.s.SWF} = 0.243\cdot kip$ 1.0E in-plane V, seismic roof diaphragm

$Vu_{i.w.SWF} := F_{7.10}\cdot 1.0v_{i.w.SW}\cdot s_{max.FSW}$ $Vu_{i.w.SWF} = 0.075\cdot kip$ 1.0W in-plane V roof diaphragm

$Vu_{o\_09dl.SWF} := \left(0.9v_{dl.FSW}\right)\cdot s_{max.FSW}$ $Vu_{o\_09dl.SWF} = 1.45\cdot kip$ 0.9D Shear from DL only

$V_{ot\_w.SWF} := \frac{v_{ot.w.FSW}}{No_{pl.FEW}}$ $V_{ot\_w.SWF} = 0\cdot kip$ 1.0W Out-of-Plane V on plates at base of endwall & end of sidewall a distance "H" from end, decoupled OT moment.

$V_{ot\_s.SWF} := \frac{v_{ot.s.FSW}}{No_{pl.FEW}}$ $V_{ot\_s.SWF} = 0\cdot kip$

$$Vu_{o.w.SWF} := F_{7.10}\cdot\max\left[\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,\begin{pmatrix}0.9v_{dl.FSW}\cdots\\+1.0\cdot v_{up.w.FSW}\end{pmatrix}\cdot s_{max.FSW}\cdots,0\cdot kip\right]$$
$$+V_{ot\_w.SWF} \qquad +-V_{ot\_w.SWF}$$

$Vu_{o.w.SWF} = 2.598\cdot kip$ 0.9D+1.0W Out-of-plane V, net uplift & decoupled OT moment. Downward forces not applicable.

$Vu_{o.s.SWF} := F_{7.10}\cdot\max\left(V_{ot\_s.SWF} - 0.9v_{dl.FSW}\cdot s_{max.FSW}, V_{ot\_s.SWF} + 0.9v_{dl.FSW}\cdot s_{max.FSW}, 0\cdot kip\right)$

$Vu_{o.s.SWF} = 1.692\cdot kip$ 0.9D+1.0E

$Vu_{o.12D.16L.SWF} := F_{7.10}\cdot\left[1.2\left(v_{dl.FSW}\right) + 1.6v_{ll.FSW}\right]\cdot s_{max.FSW}$

$Vu_{o.12D.16L.SWF} = 3.04\cdot kip$ 1.2D+1.6L V, DL + LL

Plate Calcs Floorf to Sidewall - Floor Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.9} = 5\cdot in$ $w_{pl.7.9} = 5\cdot in$ $x_{Le.7.9} = 1\cdot in$ $x_{we.7.9} = 4\cdot in$ $t_{pl.7.9} = 0.313\cdot in$

$h_{ef.7.9} = 3.719\cdot in$ $h_{a.7.9} = 8\cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.9} = 12\cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.9} = 0\cdot in$

Shear Studs: $n_{stud.7.9} = 2$ $d_{0.7.9} = 0.5\cdot in$ $A_{SE.7.9} = 0.5\cdot in^2$ $f_{uta} = 70\cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.9} = n_{stud.7.9}\cdot A_{SE.7.9}\cdot f_{uta}$ $N_{sa.7.9} = 70\cdot kip$

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.9} = 3.719 \cdot in$ $c_{a1\_po.7.9} = 10.5 \cdot in$ $c_{a2\_po.7.9} = 4 \cdot in$ $s1_{7.9} = 3 \cdot in$ $s2_{7.9} = 0 \cdot in$

Design Factors: $k_{c.7.9} = 24$ $A_{Nc.7.9} = 140.34 \cdot in^2$ $A_{Nco.7.9} = 124.48 \cdot in^2$ $e'_{N.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.9} = 1.00$ $\psi_{ed\_N.7.9} = 0.92$ $\psi_{c\_N.7.9} = 1.00$ $\psi_{cp\_N.7.9} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.9} = 11.55 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.9} = 11.91 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.9} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.9} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.9} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.9} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.9} = n_{stud.7.9} \cdot A_{SE.7.9} \cdot f_{uta}$ $V_{sa.7.9} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.9} = 3.719 \cdot in$ $h_{min.7.9} = 4 \cdot in$ $c_{a1\_op.7.9} = 4 \cdot in$ $c_{a2\_op.7.9} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.9} = 60 \cdot in^2$ $A_{Vco\_op.7.9} = 72 \cdot in^2$ $e'_{V.7.9} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.9} = 1.00$ $\psi_{ed\_Vop.7.9} = 1.00$ $\psi_{c\_Vop.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.9} = 4.53 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.9} = 4.53 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.9a} = 0 \cdot in$ $s2_{7.9a} = 3 \cdot in$ $l_{e.7.9} = 3.719 \cdot in$ $h_{ef.7.9} = 3.719 \cdot in$ $h_{a.7.9} = 8 \cdot in$ $h_{min.7.9a} = 8 \cdot in$

$c_{a1\_ip.7.9} = 13.5 \cdot in$ $c_{a2\_ip.7.9} = 4 \cdot in$

Design Factors: $A_{Vc\_ip.7.9} = 194 \cdot in^2$ $A_{Vco\_ip.7.9} = 820 \cdot in^2$ $e'_{V.7.9a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.9} = 1.00$ $\psi_{ed\_Vip.7.9} = 0.76$ $\psi_{c\_Vip.7.9} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.9} = 28.12 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.9} = 6.06 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.9} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.9} = 11.91 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.9} = k_{cp.7.9} \cdot N_{cbg.7.9}$ $V_{cpg.7.9} = 23.83 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.9} \\ 0.75 \cdot V_{cbg\_op.7.9} \\ 0.75 \cdot V_{cbg\_ip.7.9} \\ 0.7V_{cpg.7.9} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.401 \\ 4.545 \\ 16.678 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

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Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Tensile Allowable Summary

$\phi P_{pl.F\_SW} = \min\left(\left(0.75 \cdot N_{sa.7.9} \quad 0.75 \cdot N_{cbg.7.9} \quad 0.7 \cdot N_{pn.7.9}\right)\right)$ $\phi P_{pl.F\_SW} = 8.935 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.FSW}}{\phi P_{pl.F\_SW}} = 0.054 \quad \text{or} \quad \left[\frac{Vu_{o.w.FSW}}{\left(0.75 \cdot V_{cbg\_op.7.9}\right)} + \frac{Vu_{i.w.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.78 \quad < 1.0$$

$$\frac{Tu_{o.s.FSW}}{\phi P_{pl.F\_SW}} = 0.015 \quad \text{or} \quad \left[\frac{Vu_{o.s.FSW}}{0.75 \cdot V_{cbg\_op.7.9}} + \frac{Vu_{i.s.FSW}}{\left(0.75 \cdot V_{cbg\_ip.7.9}\right)}\right] = 0.551 \quad < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.9}}\right) = 0.894 \quad < 1.0$$

$Result_{FSW}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.FSW}$ = "not required" < 1.2

$chk_{6.FSW}$ = "OK"

Plate Calcs Floor to Side Wall - Side Wall Plate

PLATE PARAMETERS:

Plate Geometry: $L_{pl.7.10} = 5 \cdot in$ $w_{pl.7.10} = 5 \cdot in$ $x_{Le.7.10} = 1 \cdot in$ $x_{we.7.10} = 2.5 \cdot in$ $t_{pl.7.10} = 0.313 \cdot in$

$h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.7.10} = 12 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.7.10} = 5 \cdot in$

Shear Studs: $n_{stud.7.10} = 2$ $d_{0.7.10} = 0.5 \cdot in$ $A_{SE.7.10} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $N_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.7.10} = 1.719 \cdot in$ $c_{a1\_po.7.10} = 10.5 \cdot in$ $c_{a2\_po.7.10} = 7.5 \cdot in$ $s1_{7.10} = 3 \cdot in$ $s2_{7.10} = 0 \cdot in$

Design Factors: $k_{c.7.10} = 24$ $A_{Nc.7.10} = 42.07 \cdot in^2$ $A_{Nco.7.10} = 26.59 \cdot in^2$ $e'_{N.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.7.10} = 1.00$ $\psi_{ed\_N.7.10} = 1.00$ $\psi_{c\_N.7.10} = 1.00$ $\psi_{cp\_N.7.10} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.7.10} = 3.63 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.7.10} = 5.74 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.7.10} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.7.10} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.7.10} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.7.10} = 56.55 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.7.10} = n_{stud.7.10} \cdot A_{SE.7.10} \cdot f_{uta}$ $V_{sa.7.10} = 70 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.7.10} = 1.719 \cdot in$ $h_{min.7.10} = 4 \cdot in$ $c_{a1\_op.7.10} = 7.5 \cdot in$ $c_{a2\_op.7.10} = 10.5 \cdot in$

Design Factors: $A_{Vc\_op.7.10} = 99 \cdot in^2$ $A_{Vco\_op.7.10} = 253 \cdot in^2$ $e'_{V.7.10} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.7.10} = 1.00$ $\psi_{ed\_Vop.7.10} = 0.98$ $\psi_{c\_Vop.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.7.10} = 9.98 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.7.10} = 4.59 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{7.10a} = 0 \cdot in$ $s2_{7.10a} = 3 \cdot in$ $l_{e.7.10} = 1.719 \cdot in$ $h_{ef.7.10} = 1.719 \cdot in$ $h_{a.7.10} = 4 \cdot in$ $h_{min.7.10a} = 4 \cdot in$

$c_{a1\_ip.7.10} = 13.5 \cdot in$ $c_{a2\_ip.7.10} = 7.5 \cdot in$

Design Factors: $A_{Vc\_ip.7.10} = 111 \cdot in^2$ $A_{Vco\_ip.7.10} = 820 \cdot in^2$ $e'_{V.7.10a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.7.10} = 1.00$ $\psi_{ed\_Vip.7.10} = 0.81$ $\psi_{c\_Vip.7.10} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.7.10} = 24.1 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.7.10} = 3.17 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.7.10} = 1.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.7.10} = 5.74 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.7.10} = k_{cp.7.10} \cdot N_{cbg.7.10}$ $V_{cpg.7.10} = 5.74 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.7.10} \\ 0.75 \cdot V_{cbg\_op.7.10} \\ 0.75 \cdot V_{cbg\_ip.7.10} \\ 0.7V_{cpg.7.10} \end{pmatrix} = \begin{pmatrix} 45.5 \\ 3.442 \\ 2.381 \\ 4.018 \end{pmatrix} \cdot kip$$

ductile steel failure in shear

concrete breakout in shear - condition A

concrete breakout in shear - condition A

concrete pryout in shear - condition B

Tensile Allowable Summary

$\phi P_{pl.SWF} = \min\left(\left(0.75 \cdot N_{sa.7.10} \quad 0.75 \cdot N_{cbg.7.10} \quad 0.7 \cdot N_{pn.7.10}\right)\right)$ $\phi P_{pl.SWF} = 4.305 \cdot kip$

Plate Summary

$$\frac{Tu_{o.w.SWF}}{\phi P_{pl.SWF}} = 0.113 \quad \text{or} \quad \frac{Vu_{o.w.SWF}}{\left(0.75 \cdot V_{cbg\_op.7.10}\right)} + \frac{Vu_{i.w.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.786 < 1.0$$

$$\frac{Tu_{o.s.SWF}}{\phi P_{pl.SWF}} = 0.031 \quad \text{or} \quad \frac{Vu_{o.s.SWF}}{0.75 \cdot V_{cbg\_op.7.10}} + \frac{Vu_{i.s.SWF}}{\left(0.75 \cdot V_{cbg\_ip.7.10}\right)} = 0.594 < 1.0$$

$$\max\left(\frac{Vu_{o.12D.16L.SWF}}{0.75 \cdot V_{cbg\_op.7.10}}\right) = 0.883 < 1.0$$

$Result_{SWF}$ = "< 0.2, full strength shear/tension permitted" $IC_{pl.SWF}$ = "not required" < 1.2

$chk_{6.SWF}$ = "OK"

$$l_{w\_Rqrd.FSW.1} := \frac{\left(\sqrt{Tu_{o.w.FSW}^2 + Vu_{o.w.FSW}^2 + Vu_{i.w.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.1} = 0.775 \cdot in$

$$l_{w\_Rqrd.FSW.2} := \frac{\left(\sqrt{Tu_{o.s.FSW}^2 + Vu_{o.s.FSW}^2 + Vu_{i.s.FSW}^2}\right) \cdot \frac{1}{1.3}}{(0.3 \cdot 70 \cdot ksi) \cdot 0.125 \cdot in}$$ $l_{w\_Rqrd.FSW.2} = 0.503 \cdot in$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$$l_{w\_Rqrd.FSW.3} := \frac{\left(\sqrt{Tu_{o.w.SWF}^2 + Vu_{o.w.SWF}^2 + Vu_{i.w.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.3} = 0.775\cdot in$$

$$l_{w\_Rqrd.FSW.4} := \frac{\left(\sqrt{Tu_{o.s.SWF}^2 + Vu_{o.s.SWF}^2 + Vu_{i.s.SWF}^2}\right)\cdot\frac{1}{1.3}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.4} = 0.503\cdot in$$

$$l_{w\_Rqrd.FSW.5} := \frac{Vu_{o.12D.16L.FSW}}{(0.3\cdot 70\cdot ksi)\cdot 0.125\cdot in} \qquad l_{w\_Rqrd.FSW.5} = 1.158\cdot in$$

$$l_{w\_Rqrd.FSW.6} := \frac{\min(w_{pl.7.9}, w_{pl.7.10})}{2} \qquad l_{w\_Rqrd.FSW.6} = 2.5\cdot in$$

$l_{w\_reqd.FSW} := \max(l_{w\_Rqrd.FSW.1}, l_{w\_Rqrd.FSW.2}, l_{w\_Rqrd.FSW.3}, l_{w\_Rqrd.FSW.4}, l_{w\_Rqrd.FSW.5}, l_{w\_Rqrd.FSW.6})$ $l_{w\_reqd.FSW} = 2.5\cdot in$

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g. Floor to Foundation

Loads

$wgt_r := (t_{r.a})\cdot\gamma_{LW}$ $wgt_r = 46.708\cdot psf$ Roof Dead Load

$wgt_w := t_w\cdot\gamma_{LW}$ $wgt_w = 39.333\cdot psf$ Wall Dead Load

$wgt_{flr}^T = (41.15 \quad 40.4 \quad 39.84 \quad 39.42 \quad 39.08 \quad 38.8 \quad 38.58 \quad 37.78 \quad 37.66)\cdot psf$ Floor Dead Load

$W_{lat} = 55.643\cdot psf$ Wind Load

$w_{llf.0} = 250\cdot psf$ $w_{llf.0.ref} = \text{"Floor Live Load Option 0"}$

$w_{llf.1} = 200\cdot psf$ $w_{llf.1.ref} = \text{"Floor Live Load Option 1"}$

$w_{llf.2} = 150\cdot psf$ $w_{llf.2.ref} = \text{"Floor Live Load Option 2"}$

$A_r := L_r\cdot W_r$ $A_r^T = (148 \quad 172 \quad 196 \quad 220 \quad 244 \quad 268 \quad 292 \quad 316 \quad 340)\,ft^2$ Area of Roof/Floor

$A_f := L_f\cdot W_f$ $A_f^T = (123.7 \quad 145.7 \quad 167.6 \quad 189.5 \quad 211.4 \quad 233.3 \quad 255.2 \quad 277.2 \quad 299.1)\,ft^2$

Overturning

$$M_R := 1\left(\begin{matrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min(H_E)\cdot wgt_w \ldots \\ + W_{ew}\cdot\min(H_E)\cdot wgt_w + L_f\cdot W_f\cdot wgt_{flr} \end{matrix}\right)\cdot\left(\frac{W_{ew}}{2}\right)$$ Allowable Resisting Moment

$M_R^T = (522 \quad 532.9 \quad 543.8 \quad 554.7 \quad 565.6 \quad 576.5 \quad 587.3 \quad 598.2 \quad 609.1)\cdot kip\cdot ft$

Wind

$$M_{OT.wind} := 0.6\cdot\left[\left|P_{wind_3}\right|\cdot(L\cdot W_r)\cdot\frac{W_r}{2} + W_{lat}\,L\cdot\max(H_E)\cdot\frac{\max(H_E)}{2}\right]$$ Overturning Moment due to Wind

$M_{OT.wind}^T = (58.4 \quad 68.2 \quad 77.9 \quad 87.6 \quad 97.4 \quad 107.1 \quad 116.9 \quad 126.6 \quad 136.3)\cdot kip\cdot ft$

$1.5\,M_{OT.wind}^T = (87.6 \quad 102.3 \quad 116.9 \quad 131.5 \quad 146.1 \quad 160.7 \quad 175.3 \quad 189.9 \quad 204.5)\cdot kip\cdot ft$

Seismic

$$M_{OT.seismic} := 0.7\,C_s\cdot\left[\begin{array}{l} L_r\cdot W_r\cdot\left(wgt_r\right)\cdot\max\left(H_E\right) + L_r\cdot W_r\cdot\left(0.25w_{llr}\right)\cdot\max\left(H_E\right) \ldots \\ +\left(2L_f\cdot\max\left(H_E\right) + 2W_{ew}\cdot\max\left(H_E\right)\right)\cdot\dfrac{wgt_w\cdot\max\left(H_E\right)}{2} \ldots \\ + L_f\cdot W_f\cdot wgt_{flr}\cdot\dfrac{2}{3}\cdot t_{floor} + L_f\cdot W_f\cdot\left(0.25w_{llf.0}\right)\cdot t_{floor} \end{array}\right]$$

Overturning Moment due to seismic

$$M_{OT.seismic}^{T} = (40.609 \quad 44.896 \quad 49.184 \quad 53.471 \quad 57.759 \quad 62.046 \quad 66.334 \quad 70.621 \quad 74.909)\cdot kip\cdot ft$$

$$1.5\,M_{OT.seismic}^{T} = (60.913 \quad 67.344 \quad 73.775 \quad 80.207 \quad 86.638 \quad 93.069 \quad 99.501 \quad 105.932 \quad 112.363)\cdot kip\cdot ft$$

$$M_{ot.w.net} := \left(1.5M_{OT.wind} - M_R\right) \qquad M_{ot.s.net} := \left(1.5M_{OT.seismic} - M_R\right)$$

$$M_{ot.w.net}^{T} = (-434.4 \quad -430.7 \quad -426.9 \quad -423.2 \quad -419.5 \quad -415.8 \quad -412.1 \quad -408.3 \quad -404.6)\cdot kip\cdot ft$$

$$M_{ot.s.net}^{T} = (-461.1 \quad -465.6 \quad -470 \quad -474.5 \quad -478.9 \quad -483.4 \quad -487.8 \quad -492.3 \quad -496.8)\cdot kip\cdot ft$$

$chk_{S7.15} := \text{if}\left(\max\left(M_{ot.w.net}, M_{ot.s.net}\right) \geq 0, \text{"NG"}, \text{"OK"}\right)$ $chk_{S7.15} = \text{"OK"}$

Sliding

$$WT := \begin{pmatrix} L_r\cdot W_r\cdot wgt_r + L_f\cdot\min\left(H_E\right)\cdot wgt_w \ldots \\ +2W_{ew}\cdot\min\left(H_E\right)\cdot wgt_w + 2L_f\cdot W_f\cdot wgt_{flr} \end{pmatrix}$$

Dead Load of Shelter

$$WT^{T} = (167.854 \quad 169.72 \quad 171.587 \quad 173.454 \quad 175.32 \quad 177.187 \quad 179.054 \quad 180.921 \quad 182.787)\cdot kip$$

$$WT_{max} := \max\left[\begin{array}{l} A_r\cdot\left(wgt_r + w_{misc.r}\right) + A_f\cdot\left(wgt_{flr} + w_{misc.f}\right) \ldots \\ +2L_f\cdot\max\left(H_E\right)\cdot\left(wgt_w + w_{misc.w}\right) \ldots \\ +2\cdot W_{ew}\cdot\max\left(H_E\right)\cdot wgt_w + \left[A_r\cdot w_{llr} + A_r\cdot\left(w_{llf.0}\right)\right] \end{array}\right]$$

Maximum of all Loading Conditions Dead Load and Live Load

$WT_{max} = 234.596\cdot kip$

Use 4x4 bearing pads, $A_{pad} := 4in\cdot 4in$ $A_{pad} = 16\cdot in^2$

$S_{bp} := 3ft$ Spacing of Bearing pads, O.C.

$$wt_{pad} := \max\left[\frac{\max(WT)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad} = 7.03\cdot kip$ Weight per Bearing Pad

$$f_{pad} := \frac{wt_{pad}}{A_{pad}}$$

$f_{pad} = 439.4\cdot psi$ Compressive Stress per Pad

$F_{pad} := 800psi$ Allowable Stress per pad

$F_P := 0.7\cdot 0.85 f'_{c.fdn}$ $F_P = 1785.1\cdot psi$ Allowable Stress of Fdn Concrete

$chk_{S7.16} := \text{if}\left(F_{pad} \geq f_{pad} \wedge F_P \geq f_{pad}, \text{"OK"}, \text{"NG"}\right)$ $chk_{S7.16} = \text{"OK"}$

$$wt_{pad.big} := \max\left[\frac{\max\left(WT_{max}\right)}{\dfrac{2\left[\left(\max(L) - 2\cdot t_w\right) + \left(W_{ew}\right)\right]}{S_{bp}}}\right]$$

$wt_{pad.big} = 9.023\cdot kip$ Weight per Bearing Pad

$f_{pad.big} := \frac{wt_{pad.big}}{A_{pad}}$ $f_{pad.big} = 563.9 \cdot psi$ Compressive Stress per Pad

$chk_{S7.17} := if\left(F_{pad} \geq f_{pad.big} \wedge F_P \geq f_{pad.big}, "OK", "NG"\right)$ $chk_{S7.17} = "OK"$

$\mu := 0.25$ Coefficient of Friction

Wind $V_{net.w.sl} := max\left[0kip, \left[\mu \cdot \left[-(WT) + 1.5\left|P_{wind_3}\right| \cdot \left(A_r\right)\right] \ldots \right.\right.$
$\left.\left. + 1.5 W_{lat} L \cdot max\left(H_E\right)\right]\right]$ $V_{net.w.sl} = 0 \cdot kip$ Net Sliding Force due to Wind.

Seismic $V_{net.s.sl} := max\left[0kip, 1.5 \cdot \left[0.7 C_s \cdot \left(\begin{array}{l} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{array}\right) + -\mu \cdot \left(\begin{array}{l} 0.25 \cdot A_f \cdot w_{llf.0} \ldots \\ + 0.25 \cdot A_r \cdot w_{llr} \ldots \\ + WT \end{array}\right)\right]\right]$

$V_{net.s.sl} = 0 \cdot kip$ Sliding Force due to Seismic, +ve value indicates sliding, -ve indicates no sliding

$chk_{S7.18} := if\left(V_{net.w.sl} > 0 \vee V_{net.s.sl} > 0, "Slide Plates Req'd", "OK"\right)$ $chk_{S7.18} = "OK"$

Slide Plate, width $w_{sl.pl} := 9in$, height $h_{sl.pl} := 9in$, thickness $t_{sl.pl} := \frac{3}{16}in$

$S_{sl.pl} := \frac{t_{sl.pl}^2 \cdot h_{sl.pl}}{6}$ $S_{sl.pl} = 0.053 \cdot in^3$ Slide Plate Section Modulus

$F_{b.sl.pl} := 36ksi \cdot \frac{1}{1.67}$ $F_{b.sl.pl} = 21.557 \cdot ksi$ Allowable Bending Stress

$M_{sl.pl} := F_{b.sl.pl} \cdot S_{sl.pl} \cdot \frac{4}{3}$ $M_{sl.pl} = 1.516 \cdot kip \cdot in$ Allowable Bending Moment

$V_{per\_in} := \frac{M_{sl.pl}}{3in}$ $V_{per\_in} = 0.505 \cdot kip$

$$No_{slide.plate} := \left\| \begin{array}{l} max\left(2, ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)\right) \text{ if } \frac{ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right)}{2} \geq 1 \\ max\left[2, \left(ceil\left(\frac{max\left(0.6V_{net.w.sl}, V_{net.s.sl}\right)}{V_{per\_in}}\right) + 1\right)\right] \text{ otherwise} \end{array} \right.$$

$No_{slide.plate} = 2$ Req'd # of Slide plates per sidewall

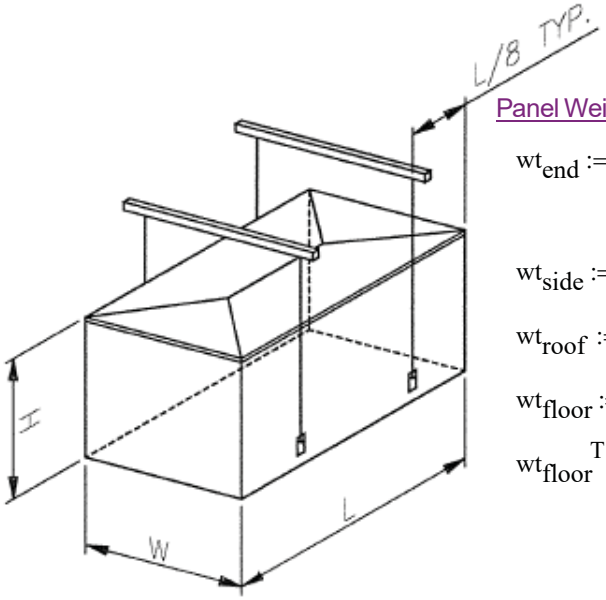
$s_{slide.pl} := \frac{L_f - 16in}{No_{slide.plate} - 1}$ Maximum Spacing of slide plates

$s_{slide.pl}^T = (10 \quad 12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26) \cdot ft$

Use a minimum spacing of $s_{slide.pl}^T = (10 \quad 12 \quad 14 \quad 16 \quad 18 \quad 20 \quad 22 \quad 24 \quad 26)\,ft$ for slide plates with a minimum of 2 per wall. Attach slide plates with (2) 1/2" DIA. exp bolts and bearing pads spaced $S_{bp} = 3\,ft$ on center.

8. SHELTER TRANSPORT

a. Analysis & Design of Shelter for Handleing & Shipping



Panel Weights

$wt_{end} := \gamma_{LW} \cdot t_w \cdot W_{ew} \cdot \max(H_E)$

$wt_{end} = 5115 \cdot lb$ Weight of End Wall

$wt_{side} := \gamma_{LW} \cdot t_w \cdot \max(H_E)$ $wt_{side} = 438 \cdot plf$ Weight of Side Wall

$wt_{roof} := \gamma_{LW} \cdot t_{r.a} \cdot W_r$ $wt_{roof} = 561 \cdot plf$ Weight of Roof

$wt_{floor} := wgt_{flr} \cdot W_{ew}$ Weight of Floor

$wt_{floor}^{T} = (480 \quad 471 \quad 465 \quad 460 \quad 456 \quad 453 \quad 450 \quad 441 \quad 439) \cdot plf$

Calculation of Stress in Side Wall Panels

$\max(w_{door}) = 6\ ft$ Maximum Door Width

$h_{door} = 7\ ft$ Maximum Door Height

$LP = 16 \cdot in$ Distance from Edge of Side Wall to Lift Point

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

The following table is the calculated handling and shipping stresses

SHELTERS

CALCULATION OF STRESS IN SIDE WALL PANELS

DUE TO HANDLING AND SHIPPING

f'c (handle)	f'c (ship)	CONC WT =	fs =	Max Door Width (ft)	Max Door Height (ft)	Lift Point (LP) ft	Load Factor Handling	Load Factor Shipping	Ft (handling, psi) = 5 (0.85) sqrt (f'_c)	Ft (shipping, psi) = 5 (0.85) sqrt (f'_c)
2475	3938	118	60000	6.00	7	1.33	1.2	1.5	211.43	266.69

Shelter Size Combinations			Stresses Checks (handling)				Stresses (handling) and checks				A_s (max, handle & ship) Required (in^2) M/(0.9f_yd)			
Width (ft)	Length (ft)	Height (ft)	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	$f_{t\ support}$ CHK	$f_{t\ door}$ CHK	$f_{t\ max+}$ CHK	$f_{t\ ad}$ CHK	A_s support	As door	As max+	As ad
12.00	12.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.016	0.042
12.00	14.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.029	0.016
12.00	16.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.044	0.011
12.00	18.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.062	0.038
12.00	20.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.082	0.065
12.00	22.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.105	0.092
12.00	24.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.131	0.119
12.00	26.00	9.00	OK	OK	OK	OK	OK	OK	OK	OK	0.012	0.050	0.159	0.146
12.00	28.00	9.00	OK	OK	OK	REINF	OK	OK	OK	REINF	0.012	0.050	0.189	0.172
12.00	9.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.000	0.009
12.00	10.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.005	0.015
12.00	10.67	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.008	0.020
12.00	18.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.058	0.029
12.00	20.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.078	0.050
12.00	22.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.100	0.071
12.00	24.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.124	0.092
12.00	26.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.150	0.113
12.00	28.00	10.00	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.047	0.179	0.133
12.00	12.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.014	0.029
12.00	14.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.026	0.011
12.00	16.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.040	0.008
12.00	18.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.056	0.026
12.00	20.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.075	0.044
12.00	22.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.096	0.062
12.00	24.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.120	0.080
12.00	26.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.145	0.099
12.00	28.00	10.67	OK	OK	OK	OK	OK	OK	OK	OK	0.011	0.045	0.173	0.117

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

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b. Lifting Plate

Load Capacity

Design for 8 1/2 ton and 12 ton

$f'_c = 4500\,psi$ — 28 Day Compressive Strength of Concrete

$f'_{c.h} = 2200\,psi$ — Compressive Strength of Concrete for Handling

$f'_{c.s} = 3500\,psi$ — Compressive Strength of Concrete for Shipping

$f'_{c.h.s} := 0.75 \cdot f'_c$ $\quad f'_{c.h.s} = 3375\,psi$ — 7-Day Compressive Strength of Concrete

8 1/2 Ton Design

$P_{8.5ton} := 8.5ton$ $\quad P_{8.5ton} = 17 \cdot kip$ — Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ — Lifting Angle, Relative to Horizontal

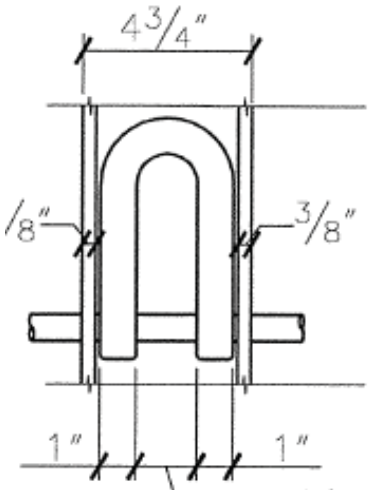
$LF_{s.h} := 1.2$ $\quad LF_{s.h} = 1.2$ — Load Factor for Handling

$R_{8.5t} := LF_{s.h} \cdot \frac{P_{8.5ton}}{\sin(\theta_{lift})}$ $\quad R_{8.5t} = 21.7 \cdot kip$ — Factored Resultant Load for Rod & Clevis

$V_{8.5t.horz} := R_{8.5t} \cdot \cos(\theta_{lift})$ $\quad V_{8.5t.horz} = 7.42 \cdot kip$ — Resultant Horizontal Shear Force

$V_{8.5t.vert} := R_{8.5t} \cdot \sin(\theta_{lift})$ $\quad V_{8.5t.vert} = 20.4 \cdot kip$ — Resultant Vertical Shear Force

Rod & Clevis:



Use 1" (8 1/2 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{8.5t} := \frac{R_{8.5t}}{2}$ $\quad V_{8.5t} = 10.85 \cdot kip$

$M_{8.5t} := V_{8.5t} \cdot \frac{(4in - 3.6875in)}{2}$ $\quad M_{8.5t} = 1.7 \cdot kip \cdot in$

$d_{8.5t.bar} := 1in$

$A_{8.5t.bar} := \frac{\pi \cdot d_{8.5t.bar}^2}{4}$ $\quad A_{8.5t.bar} = 0.7854 \cdot in^2$

$I_{8.5t.bar} := \frac{\pi}{4} \cdot (0.5 \cdot d_{8.5t.bar})^4$ $\quad I_{8.5t.bar} = 0.0491 \cdot in^4$

$S_{8.5t.bar} := \frac{I_{8.5t.bar}}{d_{8.5t.bar} \cdot 0.5}$ $\quad S_{8.5t.bar} = 0.0982 \cdot in^3$

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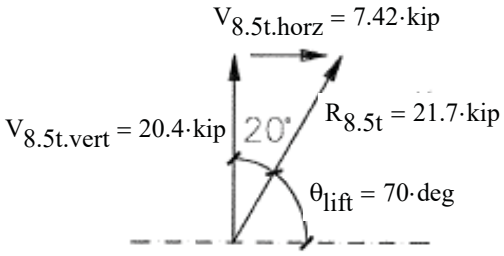
Manufacturer:	Modular Connections, LLC



$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.8.5t} := \frac{V_{8.5t}}{A_{8.5t.bar}}$ $f_{v.8.5t} = 13.82 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$f_{b.8.5t} := \frac{M_{8.5t}}{S_{8.5t.bar}}$ $f_{b.8.5t} = 17.28 \cdot ksi$ 8.5 ton Lift Plate Applied Shear Stress

$chk_{8.5t.rod.stress} := if\left(\frac{f_{v.8.5t}}{F_v} \le 1.0 \wedge \frac{f_{b.8.5t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{8.5t.rod.stress} = "OK"$

Use A570 Grade 50 1" diameter rod

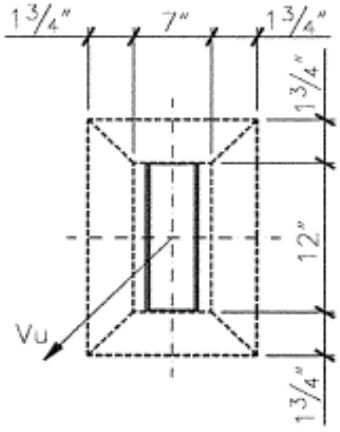
Shear Capacity

Horizontal Shear

$\lambda = 0.85$

Classify lifting plate as an insert.

$Vu_{8.5t.horz} := 1.4 \cdot V_{8.5t.horz}$ $Vu_{8.5t.horz} = 10.39 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.8.5t} := if\left(Vu_{8.5t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.8.5t} = "OK"$

$Vu_{8.5t.vert} := 1.4 \cdot V_{8.5t.vert}$ $Vu_{8.5t.vert} = 28.56 \cdot kip$

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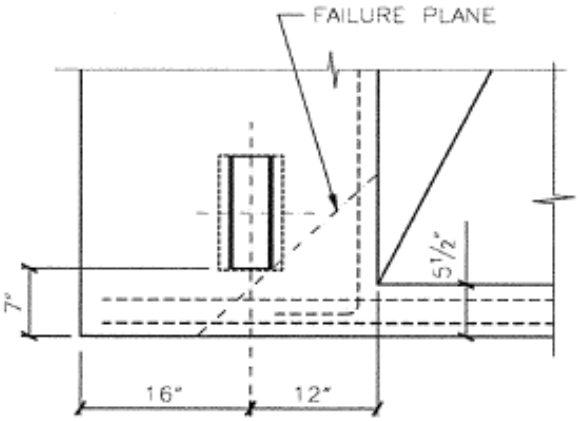
Vertical Shear

Classify lifting plate as an insert.



Shear Friction:

$A_{v.0.1} := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.625in + 7in)$

$A_{v.0.1} = 0.79 \cdot in^2$

$\mu_{0.1} := 1.4 \cdot \lambda$ $\mu_{0.1} = 1.2$

$\phi Vn_{1.0.1} := A_{v.0.1} \cdot f_y \cdot \left(\mu_{0.1} \cdot \sin(45deg) + \cos(45deg)\right)$

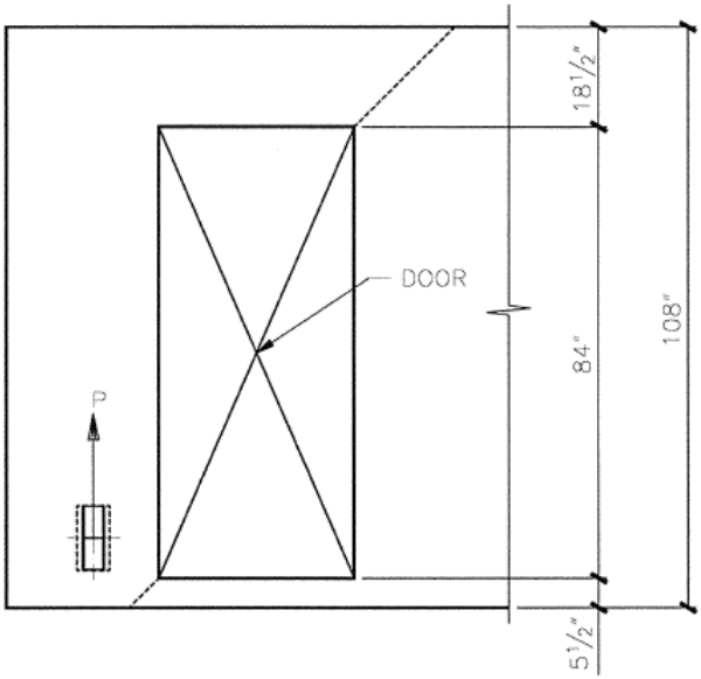
$\phi Vn_{1.0.1} = 73.05 \cdot kip$

$\phi Vn_{2.0.1} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.625in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_{2.0.1} = 63.48 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.1} := \phi_V \min\left(\phi Vn_{1.0.1}, \phi Vn_{2.0.1}\right)$ $\phi Vn_{0.1} = 47.61 \cdot kip$

$chk_{v.v.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.1}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.v.8.5t} = \text{"OK"}$



$A_{v.0.2} := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (18.5in)$

$A_{v.0.2} = 1.24 \cdot in^2$

$\mu_{0.2} := 1.4 \cdot \lambda$ $\mu_{0.2} = 1.2$

$\phi Vn_{1.0.2} := A_{v.0.2} \cdot f_y \cdot \left(\mu_{0.2} \cdot \sin(45deg) + \cos(45deg)\right)$

$\phi Vn_{1.0.2} = 115.68 \cdot kip$

$\phi Vn_{2.0.2} := 0.2 \cdot f'_{c.h.s} \cdot \frac{(18.5in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_{2.0.2} = 70.64 \cdot kip$

$\phi_V = 0.75$

$\phi Vn_{0.2} := \phi_V \min\left(\phi Vn_{1.0.2}, \phi Vn_{2.0.2}\right)$

$\phi Vn_{0.2} = 52.98 \cdot kip$

$chk_{v.2.8.5t} := \text{if}\left(Vu_{8.5t.vert} \le \phi Vn_{0.2}, \text{"OK"}, \text{"NG"}\right)$ $chk_{v.2.8.5t} = \text{"OK"}$

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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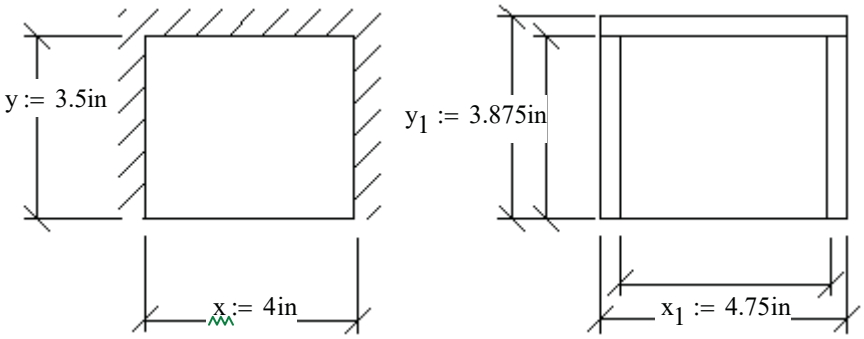
Bearing Pressure

Vertical



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Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

$f'_{c.h.s} = 3375\,psi$

$A_{reqd.8.5t.bp} := \frac{1.4 \cdot V_{8.5t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.bp} = 14.2 \cdot in^2$

$A_{prov.8.5t.bp} := x_1 \cdot y_1$ $A_{prov.8.5t.bp} = 18.41 \cdot in^2$

$chk_{8.5t.bp.area} := if\left(A_{prov.8.5t.bp} \geq A_{reqd.8.5t.bp}, "OK", "NG"\right)$ $chk_{8.5t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $b := y$ $\frac{a}{b} = 1.1$ $t := \frac{5}{8}in$ Thickness of Plate

$f_{p.8.5t} := \frac{V_{8.5t.vert}}{a \cdot b}$ $f_{p.8.5t} = 1.46 \cdot ksi$ Plate Bearing Pressure

---

$\beta = 0.373$

$\sigma := \frac{\beta \cdot f_{p.8.5t} \cdot b^2}{t^2}$ $\sigma = 17.1 \cdot ksi$ Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.bearing} := if\left(\sigma_{allow} \geq \sigma, "OK", "NG"\right)$ $chk_{8.5t.bearing} = "OK"$

Horizontal

$P_{8.5t.horz} := 1.4V_{8.5t.horz}$ $P_{8.5t.horz} = 10.4 \cdot kip$

$A_{reqd.8.5t.vert} := \frac{P_{8.5t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.8.5t.vert} = 5.2 \cdot in^2$

$A_{prov.8.5t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.8.5t.vert} = 27 \cdot in^2$

$chk_{8.5t.vert.area} := if\left(A_{prov.8.5t.vert} \geq A_{reqd.8.5t.vert}, "OK", "NG"\right)$ $chk_{8.5t.vert.area} = "OK"$

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$f_{p.8.5t} := \frac{P_{8.5t.horz}}{A_{prov.8.5t.vert}}$ $f_{p.8.5t} = 0.38 \cdot ksi$ Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.b.horz} := if(\sigma_{allow} \geq f_{p.8.5t}, "OK", "NG")$ $chk_{8.5t.b.horz} = "OK"$

$t := \frac{3}{8} in$ Thickness of Plate Carring Bending

$M_{8.5t.horz} := \frac{(f_{p.8.5t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{8.5t.horz} = 0.65 \cdot kip \cdot in$

$S_{8.5t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{8.5t.horz} = 1 \cdot in^3$

$f_{b.8.5t.horz} := \frac{M_{8.5t.horz}}{S_{8.5t.horz}}$ $f_{b.8.5t.horz} = 0.6 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{8.5t.horz.bend} := if(\sigma_{allow} \geq f_{b.8.5t.horz}, "OK", "NG")$ $chk_{8.5t.horz.bend} = "OK"$

Welds

Vertical

1" Diameter Bar to Side Plate

$P := \frac{R_{8.5t}}{2}$ $P = 10.85 \cdot kip$

$Circ_{8.5t} := \pi \cdot d_{8.5t.bar}$ $Circ_{8.5t} = 3.14 \cdot in$

$f_{w.8.5t.bar} := \frac{P}{Circ_{8.5t}}$ $f_{w.8.5t.bar} = 3.46 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.weld} := if(F_w \geq f_{w.8.5t.bar}, "OK", "NG")$ $chk_{8.5t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{8.5t.vert}}{2}$ $P = 10.2 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.TP} := \frac{P}{l}$ $f_{w.8.5t.SP.TP} = 2.91 \cdot \frac{kip}{in}$

$t_{weld} := 0.25in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 3.7 \cdot \frac{kip}{in}$ Strength of weld

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$chk_{8.5t.bar.tp.weld} := if\left(F_w \geq f_{w.8.5t.SP.TP}, "OK", "NG"\right)$ $chk_{8.5t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{8.5t.horz}}{2}$ $P = 3.71 \cdot kip$

$l := 12in$ $l = 3.5 \cdot in$

$f_{w.8.5t.SP.BP} := \frac{P}{l}$ $f_{w.8.5t.SP.BP} = 0.31 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_w := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{8.5t.bar.bck.weld} := if\left(F_w \geq f_{w.8.5t.SP.BP}, "OK", "NG"\right)$ $chk_{8.5t.bar.bck.weld} = "OK"$

12 Ton Design

$P_{12ton} := 12ton$ $P_{12ton} = 24 \cdot kip$ Maximum Load for Rod & Clevis

$\theta_{lift} := 70deg$ Lifting Angle, Relative to Horizontal

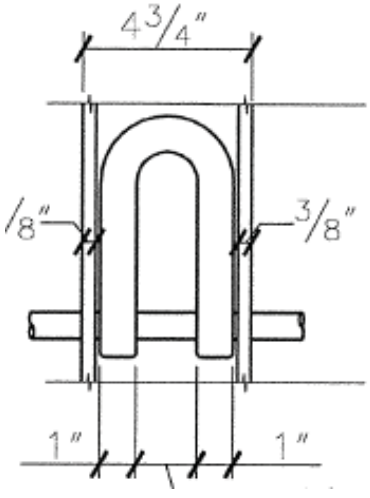
$LF_{s.h} := 1.2$ $LF_{s.h} = 1.2$ Load Factor for Handling

$R_{12t} := LF_{s.h} \cdot \frac{P_{12ton}}{\sin\left(\theta_{lift}\right)}$ $R_{12t} = 30.6 \cdot kip$ Factored Resultant Load for Rod & Clevis

$V_{12t.horz} := R_{12t} \cdot \cos\left(\theta_{lift}\right)$ $V_{12t.horz} = 10.48 \cdot kip$ Resultant Horizontal Shear Force

$V_{12t.vert} := R_{12t} \cdot \sin\left(\theta_{lift}\right)$ $V_{12t.vert} = 28.8 \cdot kip$ Resultant Vertical Shear Force

Rod & Clevis:



Use 1 1/4" (12 ton) clevis
Re: McMaster's Catalog pg.426 (1989)

$V_{12t} := \frac{R_{12t}}{2}$ $V_{12t} = 15.32 \cdot kip$

$M_{12t} := V_{12t} \cdot \frac{(4in - 3.6875in)}{2}$ $M_{12t} = 2.39 \cdot kip \cdot in$

$d_{12t.bar} := 1.25in$

$A_{12t.bar} := \frac{\pi \cdot d_{12t.bar}^2}{4}$ $A_{12t.bar} = 1.2272 \cdot in^2$

$I_{12t.bar} := \frac{\pi}{4} \cdot \left(0.5 \cdot d_{12t.bar}\right)^4$ $I_{12t.bar} = 0.1198 \cdot in^4$

$S_{12t.bar} := \frac{I_{12t.bar}}{d_{12t.bar} \cdot 0.5}$ $S_{12t.bar} = 0.1917 \cdot in^3$

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$V_{12t.horz} = 10.48 \cdot kip$

$V_{12t.vert} = 28.8 \cdot kip$ 20° $R_{12t} = 30.6 \cdot kip$

$\theta_{lift} = 70 \cdot deg$

$F_{u.lift} = 65 \cdot ksi$ Tensile Stress of Rod

$F_{y.lift} = 50 \cdot ksi$ Yield Stress of Rod

$F_v := 0.22 \cdot F_{u.lift}$ $F_v = 14.3 \cdot ksi$ Allowable Shear Stress of Rod

$F_b := 0.75 \cdot F_{u.lift}$ $F_b = 48.75 \cdot ksi$ Allowable Bending Stress of Rod

$f_{v.12t} := \frac{V_{12t}}{A_{12t.bar}}$ $f_{v.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$f_{b.12t} := \frac{M_{12t}}{S_{12t.bar}}$ $f_{b.12t} = 12.49 \cdot ksi$ 8 ton Lift Plate Applied Shear Stress

$chk_{12t.rod.stress} := if\left(\frac{f_{v.12t}}{F_v} \le 1.0 \wedge \frac{f_{b.12t}}{F_b} \le 1.0, "OK", "NG"\right)$ $chk_{12t.rod.stress} = "OK"$

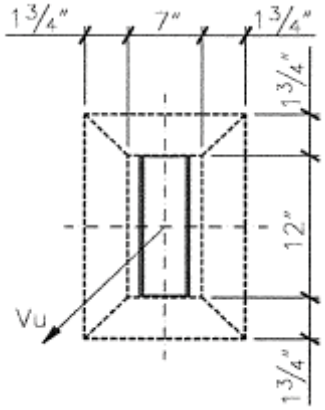
Use A570 Grade 50 1 1/4" diameter rod

Shear Capacity

Horizontal Shear

Classify lifting plate as an insert.

$Vu_{12t.horz} := 1.4 \cdot V_{12t.horz}$ $Vu_{12t.horz} = 14.68 \cdot kip$



$\phi Vc := 0.85 \cdot \lambda \cdot 4 \cdot \sqrt{f'_{c.h.s} \cdot psi} \cdot 3.5in \cdot (8.75in + 13.75in) \cdot 2$ $\phi Vc = 26.44 \cdot kip$

$chk_{v.h.12t} := if\left(Vu_{12t.horz} \le \phi Vc, "OK", "NG"\right)$ $chk_{v.h.12t} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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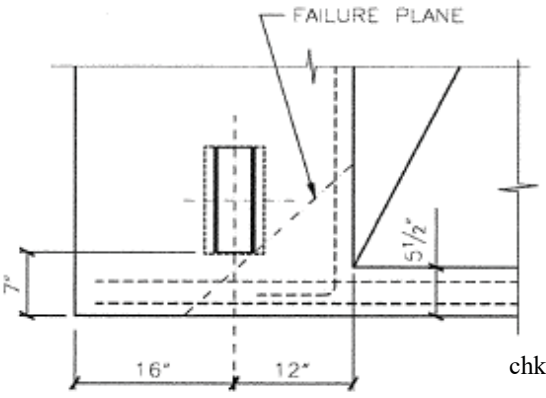
Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Vertical Shear

Classify lifting plate as an insert.

$Vu_{12t.vert} := 1.4 \cdot V_{12t.vert}$ $Vu_{12t.vert} = 40.32 \cdot kip$

Shear Friction:



$A_v := 2A_5 + WWF_{4x4\_w4xw4} \cdot (9.25in + 7in)$ $A_v = 0.78 \cdot in^2$

$\mu := 1.4 \cdot \lambda$ $\mu = 1.2$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$ $\phi Vn_1 = 72.71 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h.s} \cdot \frac{(7in + 9.25in)}{\sin(45deg)} \cdot t_w$ $\phi Vn_2 = 62.05 \cdot kip$

$\phi_V = 0.75$

$\phi Vn := \phi_V \min\left(\phi Vn_1, \phi Vn_2\right)$ $\phi Vn = 46.54 \cdot kip$

$chk_{v.v.12t} := if\left(Vu_{12t.vert} \le \phi Vn, "OK", "NG"\right)$ $chk_{v.v.12t} = "OK"$

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Allowable No. of Floors:	One (1)
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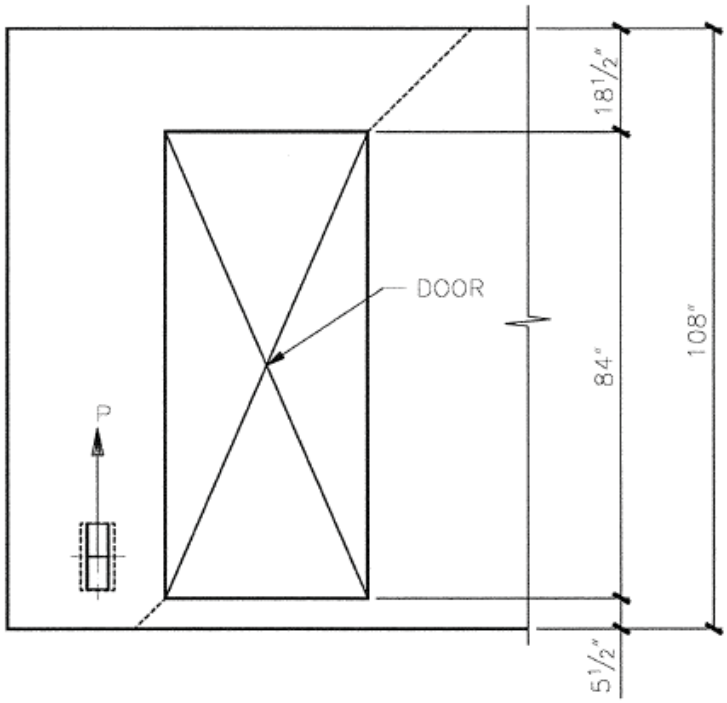


$A_w := 2 \cdot A_5 + A_6 + WWF_{4x4\_w4xw4} \cdot (112in)$

$A_v = 2.18 \cdot in^2$

$\mu := 1.4 \cdot \lambda$

$\phi Vn_1 := A_v \cdot f_y \cdot (\mu \cdot \sin(45deg) + \cos(45deg))$

$\phi Vn_1 = 202.55 \cdot kip$

$\phi Vn_2 := 0.2 \cdot f'_{c.h} \cdot \frac{(112in)}{\sin(45deg)} \cdot t_w$

$\phi Vn_2 = 278.78 \cdot kip$

$\phi_V = 0.75$

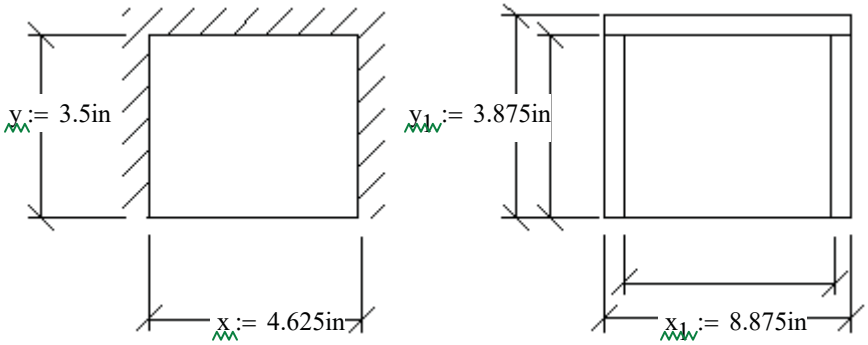
$\phi Vn := \phi_V \min(\phi Vn_1, \phi Vn_2)$

$\phi Vn = 151.91 \cdot kip$

$chk_{v.2.12t} := if(Vu_{12t.vert} \le \phi Vn, "OK", "NG")$

$chk_{v.2.12t} = "OK"$

Bearing Pressure Vertical



$f'_{c.h.s} = 3375\,psi$

$A_{reqd.12t.bp} := \frac{1.4 \cdot V_{12t.vert}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $\quad A_{reqd.12t.bp} = 20.1 \cdot in^2$

$A_{prov.12t.bp} := x_1 \cdot y_1$ $\quad A_{prov.12t.bp} = 34.39 \cdot in^2$

$chk_{12t.bp.area} := if(A_{prov.12t.bp} \ge A_{reqd.12t.bp}, "OK", "NG")$ $\quad chk_{12t.bp.area} = "OK"$

From Roak's Formulas for Stress and Strain, 7th ed. by Warren C. Young and Richard G. Budynas, page 510.

$a := x$ $\quad b := y$ $\quad \frac{a}{b} = 1.3$ $\quad t := \frac{5}{8}in$ Thickness of Plate

$f_{p.12t} := \frac{V_{12t.vert}}{a \cdot b}$ $\quad f_{p.12t} = 1.78 \cdot ksi$ Plate Bearing Pressure

$\beta = 0.456$

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$\sigma := \frac{\beta \cdot f_{p.12t} \cdot b^2}{t^2}$ $\sigma = 25.5 \cdot ksi$

Plate Stress

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.bearing} := if(\sigma_{allow} \geq \sigma, "OK", "NG")$ $chk_{12t.bearing} = "OK"$

Horizontal

$P_{12t.horz} := 1.4V_{12t.horz}$ $P_{12t.horz} = 14.7 \cdot kip$

$A_{reqd.12t.vert} := \frac{P_{12t.horz}}{0.7 \cdot \lambda \cdot f'_{c.h.s}}$ $A_{reqd.12t.vert} = 7.3 \cdot in^2$

$A_{prov.12t.vert} := (1.125in) \cdot (12in) \cdot 2$ $A_{prov.12t.vert} = 27 \cdot in^2$

$chk_{12t.vert.area} := if(A_{prov.12t.vert} \geq A_{reqd.12t.vert}, "OK", "NG")$ $chk_{12t.vert.area} = "OK"$

$f_{p.12t} := \frac{P_{12t.horz}}{A_{prov.12t.vert}}$ $f_{p.12t} = 0.54 \cdot ksi$

Plate Bearing Pressure

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.b.horz} := if(\sigma_{allow} \geq f_{p.12t}, "OK", "NG")$ $chk_{12t.b.horz} = "OK"$

$t := \frac{3}{8}in$

Thickness of Plate Carring Bending

$M_{12t.horz} := \frac{(f_{p.12t} \cdot 1.125in) \cdot 12in^2}{8}$ $M_{12t.horz} = 0.917 \cdot kip \cdot in$

$S_{12t.horz} := \frac{t \cdot (4in)^2}{6}$ $S_{12t.horz} = 1 \cdot in^3$

$f_{b.12t.horz} := \frac{M_{12t.horz}}{S_{12t.horz}}$ $f_{b.12t.horz} = 0.9 \cdot ksi$

$\sigma_{allow} := 0.75 \cdot 36ksi$ $\sigma_{allow} = 27 \cdot ksi$

$chk_{12t.horz.bend} := if(\sigma_{allow} \geq f_{b.12t.horz}, "OK", "NG")$ $chk_{12t.horz.bend} = "OK"$

Welds

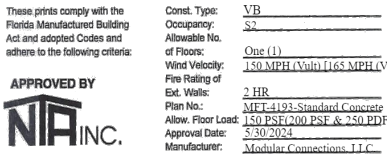
Vertical

1 1/4" Diameter Bar to Side Plate

$P := \frac{R_{12t}}{2}$ $P = 15.32 \cdot kip$

$Circ_{12t} := \pi \cdot d_{12t.bar}$ $Circ_{12t} = 3.93 \cdot in$

$f_{w.12t.bar} := \frac{P}{Circ_{12t}}$ $f_{w.12t.bar} = 3.9 \cdot \frac{kip}{in}$



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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.weld} := if(F_w \geq f_{w.12t.bar}, "OK", "NG")$ $chk_{12t.bar.weld} = "OK"$

Side Plate to Top Plate

$P := \frac{V_{12t.vert}}{2}$ $P = 14.4 \cdot kip$

$l := y$ $l = 3.5 \cdot in$

$f_{w.12t.SP.TP} := \frac{P}{l}$ $f_{w.12t.SP.TP} = 4.11 \cdot \frac{kip}{in}$

$t_{weld} := \frac{5}{16}in$ Fillet Weld Thickness

$F_{yw} := 70ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 4.6 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.tp.weld} := if(F_w \geq f_{w.12t.SP.TP}, "OK", "NG")$ $chk_{12t.bar.tp.weld} = "OK"$

Horizontal

Side Plate to Back Plate

$P := \frac{V_{12t.horz}}{2}$ $P = 5.24 \cdot kip$

$l := 12in$ $l = 12 \cdot in$

$f_{w.12t.SP.BP} := \frac{P}{l}$ $f_{w.12t.SP.BP} = 0.44 \cdot \frac{kip}{in}$

$t_{weld} := \frac{3}{16}in$ Fillet Weld Thickness

$F_{yw} = 70 \cdot ksi$ Weld Rupture Strength for Electrode grade Weld = "E70XX"

$F_{ww} := 0.3 \cdot F_{yw} \cdot 0.707 \cdot t_{weld}$ $F_w = 2.8 \cdot \frac{kip}{in}$ Strength of weld

$chk_{12t.bar.bck.weld} := if(F_w \geq f_{w.12t.SP.BP}, "OK", "NG")$ $chk_{12t.bar.bck.weld} = "OK"$

Dead Loads:

$DL_r := \max(wt_{roof}) \cdot \max(L)$ $DL_r = 15.7 \cdot kip$ Roof Dead Load

$DL_{fl} := \max(wt_{floor}) \cdot \max(L)$ $DL_{fl} = 13.4 \cdot kip$ Floor Dead Load

$w_{DL.w} := 2\max(wt_{side}) \cdot \max(L)$ $w_{DL.w} = 24.6 \cdot kip$ Wall Dead Load

$w_{ew} := 2\max(wt_{end})$ $w_{ew} = 10.2 \cdot kip$ Weight of End Wall

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APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

$DL := DL_r + DL_{fl} + w_{DL.w} + w_{ew}$ $DL = 63.9 \cdot kip$ Factored Dead Load of Shelter

$$CG := \frac{\max(H_E) \cdot DL_r + t_{floor} \cdot DL_{fl}}{DL_r + DL_{fl}} \quad CG = 6.3\,ft$$ Center of gravity from bottom of shelter

Tie Down Bracket $Impact := 1.4 \cdot 1.3$

Methodology: Prevent sliding & overturning during transport. If shelter does not slide or overturn, the tie-downs are sufficient, ramp super-elevation is neglected, conservate approach.

$wt := Impact\,DL$ $wt = 116.3 \cdot kip$ Max Shelter Weight

$Vel := 35mph$ Velocity of Truck (Exit Ramp)

$r_{ramp} := 350\,ft$ Radius of Exit Ramp

$$F_{turn} := \frac{\frac{wt}{g} \cdot Vel^2}{r_{ramp}} \quad F_{turn} = 27.2 \cdot kip$$ Total Lateral Force during Turning

$W_{trailer} := 8.5ft$ Width of Trailer

$M_{OT.Trans} := F_{turn} \cdot (CG + 4in)$ $M_{OT.Trans} = 180.9 \cdot kip \cdot ft$ Overtruning Moment

$$M_{OT.Resist} := 0.9 \cdot DL \cdot \frac{W_{trailer}}{2} \quad M_{OT.Resist} = 244.5 \cdot kip \cdot ft$$ Resisting Overturning Moment

$M_{net.trans} := M_{OT.Trans} - M_{OT.Resist}$ $M_{net.trans} = -63.6 \cdot kip \cdot ft$ Net overturning moment (-ve no overturning)

$$F_{OT.trans} := \frac{M_{net.trans}}{W_{trailer}} \quad F_{OT.trans} = -7.5 \cdot kip$$ Uplift force due to overturning (1 side)

$\mu_{trans} := 0.2$ Coefficient of Friction, sliding on trailer

$F_{net.slide.trans} := F_{turn} - \mu_{trans}\,DL$ $F_{net.slide.trans} = 14.4 \cdot kip$ Net sliding force, transport (-ve no sliding)

$No_{tie} := 2$ Number of Tie downs per side

$\theta := 45deg$ Minimum angle of Tie down to trailer

$$V_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \cos(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \cos(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$V_{tie.max} = 7.2 \cdot kip$

Maximum Shear Force Per Tie-Down, θ_{min}

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$$T_{tie.max} := \begin{cases} \frac{F_{OT.trans} \cdot \sin(\theta)}{No_{tie}} & \text{if } F_{net.slide.trans} < 0kip \wedge F_{OT.trans} > 0kip \\ \frac{F_{net.slide.trans}}{No_{tie}} & \text{if } F_{OT.trans} < 0kip \wedge F_{net.slide.trans} > 0kip \\ 0kip & \text{if } F_{OT.trans} = 0kip \wedge F_{net.slide.trans} = 0kip \\ \frac{F_{OT.trans} \cdot \sin(\theta) + F_{net.slide.trans}}{No_{tie}} & \text{otherwise} \end{cases}$$

$T_{tie.max} = 7.2 \cdot kip$

Maximum Tensile Force Per Tie-Down, θ_{min}

Plate Calcs - Tie Down Plate

PLATE PARAMETERS: : (Plate #24)

Plate Geometry: $L_{pl.8.1} = 17 \cdot in$ $w_{pl.8.1} = 5 \cdot in$ $x_{Le.8.1} = 1.5 \cdot in$ $x_{we.8.1} = 1.5 \cdot in$ $t_{pl.8.1} = 0.375 \cdot in$

$h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$

Parallel to Row of Plates: Distance from edge of panel to center of plate $x_{es\_s.8.1} = 24 \cdot in$

Perp. to Row of Plates: Edge Distance from edge of panel to front edge of plate $x_{es\_e.8.1} = 4 \cdot in$

Shear Studs: $n_{stud.8.1} = 4$ $d_{0.8.1} = 0.5 \cdot in$ $A_{SE.8.1} = 0.5 \cdot in^2$ $f_{uta} = 70 \cdot ksi$

TENSION CAPACITIES:

STEEL STRENGTH OF ANCHOR IN TENSION

Nominal Strength in Tension of Group of Anchors $N_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $N_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $h_{ef.8.1} = 5.7 \cdot in$ $c_{a1\_po.8.1} = 17 \cdot in$ $c_{a2\_po.8.1} = 5.5 \cdot in$ $s1_{8.1} = 14 \cdot in$ $s2_{8.1} = 2 \cdot in$

Design Factors: $k_{c.8.1} = 24$ $A_{Nc.8.1} = 537.9 \cdot in^2$ $A_{Nco.8.1} = 294.36 \cdot in^2$ $e'_{N.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_N.8.1} = 1.00$ $\psi_{ed\_N.8.1} = 0.89$ $\psi_{c\_N.8.1} = 1.00$ $\psi_{cp\_N.8.1} = 1.00$ cracking occurs, cast-in anchors

Basic Breakout Strength $N_{b.8.1} = 22.02 \cdot kip$

Nominal Concrete Breakout Strength in Tension of a Group of Anchors $N_{cbg.8.1} = 35.9 \cdot kip$

PULLOUT STRENGTH OF ANCHOR IN TENSION

Plate Parameters: $A_{brg.8.1} = 0.785 \cdot in^2$ Adjustment Factors: $\psi_{c\_P.8.1} = 1.00$ cracking may occur

Pullout Strength in Tension of Single-Headed Stud $N_{p.8.1} = 28.27 \cdot kip$

Nominal Pullout Strength in Tension of a Group of Anchors $N_{pn.8.1} = 113.1 \cdot kip$

SHEAR CAPACITIES:

STEEL STRENGTH OF ANCHOR IN SHEAR

Nominal Strength in Shear of Group of Anchors $V_{sa.8.1} = n_{stud.8.1} \cdot A_{SE.8.1} \cdot f_{uta}$ $V_{sa.8.1} = 140 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (Out-of-plane)

Plate Parameters: $l_{e.8.1} = 4 \cdot in$ $h_{min.8.1} = 7.5 \cdot in$ $c_{a1\_op.8.1} = 7.5 \cdot in$ $c_{a2\_op.8.1} = 17 \cdot in$

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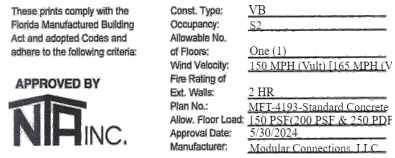
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC



Design Factors: $A_{Vc\_op.8.1} = 274 \cdot in^2$ $A_{Vco\_op.8.1} = 253 \cdot in^2$ $e'_{V.8.1} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vop.8.1} = 1.00$ $\psi_{ed\_Vop.8.1} = 1.00$ $\psi_{c\_Vop.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_op.8.1} = 11.81 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_op.8.1} = 15.33 \cdot kip$

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR FOR RESISTING "V" (In-plane)

Plate Parameters: $s1_{8.1a} = 2 \cdot in$ $s2_{8.1a} = 14 \cdot in$ $l_{e.8.1} = 4 \cdot in$ $h_{ef.8.1} = 5.719 \cdot in$ $h_{a.8.1} = 8 \cdot in$ $h_{min.8.1a} = 8 \cdot in$

$c_{a1\_ip.8.1} = 31 \cdot in$ $c_{a2\_ip.8.1} = 5.5 \cdot in$

Design Factors: $A_{Vc\_ip.8.1} = 432 \cdot in^2$ $A_{Vco\_ip.8.1} = 4324 \cdot in^2$ $e'_{V.8.1a} = 0 \cdot in$

Adjustment Factors: $\psi_{ec\_Vip.8.1} = 1.00$ $\psi_{ed\_Vip.8.1} = 0.74$ $\psi_{c\_Vip.8.1} = 1.20$ cracking, edge reinf provided

Basic Breakout Strength $V_{b\_ip.8.1} = 99.28 \cdot kip$

Nominal Concrete Breakout Strength in Shear of a Group of Anchors $V_{cbg\_ip.8.1} = 8.75 \cdot kip$

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

Design Factors: $k_{cp.8.1} = 2.00$

The pryout resistance is approximated by 1-2 times the anchor tensile resistance.

Nominal Concrete Breakout Strength in Tension of a Group of Anchors: $N_{cbg.8.1} = 35.9 \cdot kip$

Nominal Concrete Pryout Strength for a Group of Anchors: $V_{cpg.8.1} = k_{cp.8.1} \cdot N_{cbg.8.1}$ $V_{cpg.8.1} = 71.81 \cdot kip$

PLATE SUMMARY:

Shear Allowable Summary

$$\begin{pmatrix} 0.65V_{sa.8.1} \\ 0.75 \cdot V_{cbg\_op.8.1} \\ 0.75 \cdot V_{cbg\_ip.8.1} \\ 0.7V_{cpg.8.1} \end{pmatrix} = \begin{pmatrix} 91 \\ 11.499 \\ 6.565 \\ 50.266 \end{pmatrix} \cdot kip$$

ductile steel failure in shear
concrete breakout in shear - condition A
concrete breakout in shear - condition A
concrete pryout in shear - condition B

Tensile Allowable Summary

$$\phi P_{pl.FlrTie} = \min\left(\left(0.75 \cdot N_{sa.8.1} \quad 0.75 \cdot N_{cbg.8.1} \quad 0.7 \cdot N_{pn.8.1}\right)\right) \qquad \phi P_{pl.FlrTie} = 26.928 \cdot kip$$

Plate Summary $\dfrac{T_{tie.max}}{\phi P_{pl.FlrTie}} = 0.268$ or $\dfrac{V_{tie.max}}{0.75 \cdot V_{cbg\_op.8.1}} = 0.628$ < 1.0

$$\max\left(\frac{T_{.tie.max}}{\phi P_{.pl.FlrTie}}\right) + \frac{V_{.tie.max}}{0.75 \cdot V_{.cbg\_op.8.1}} = 0.896$$

$Result_{FlrTie}$ = "> 0.2, therefore interaction equation must be used" $IC_{pl.FlrTie} = 0.896$ < 1.2

$chk_{8.Flr.Tie}$ = "OK"

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9. FIRE RATING & THERMAL EFFICIENCY

IBC Table 721.1(2) 4 requires 3.8" thick sand-lightweight concrete for a 2 hour fire rating.

The shelter has a minimum 4" thick roof and walls. Therefore the 2 hour fire rating is provided in the roof and wall members.

Material Ratings

$$Comp = \begin{pmatrix} \text{"Outside Air Film"} & 2.5 \\ \text{"Sand LW Concrete"} & 0.45 \\ \text{"Foam Board Insulation"} & 5 \\ \text{"Tuff R Insulation"} & 6.6 \\ \text{"Plywood"} & 1.24 \\ \text{"Inside Air Film"} & 6.8 \\ \text{"Polyethylene 20mm"} & 0.5 \\ \text{"Wall Air Space Reflect"} & 5.32 \end{pmatrix} \cdot \frac{1}{in}$$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Insulaton Composition
Material Ratings

a. Wal

$$Wall_{2.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 0.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{2.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 0.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.2.25} := diag\left(Wall_{2.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.2.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 3.3 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W2.25} := \sum\left(R_{W.2.25}\right)$$

$$U_{W2.25} := \frac{1}{R_{W2.25}}$$

$R_{W2.25} = 13.3$ $U_{W2.25} = 0.075$

$$Wall_{3.25} := \begin{pmatrix} 0.1in \\ t_w \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0.02in \\ 0.25in \end{pmatrix} \quad Wall_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0.02 \\ 0.25 \end{pmatrix} \cdot in$$

$$R_{W.3.25} := diag\left(Wall_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \qquad R_{W.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0.01 \\ 1.33 \end{pmatrix}$$

$$R_{W3.25} := \sum\left(R_{W.3.25}\right)$$

$$U_{W3.25} := \frac{1}{R_{W3.25}}$$

$R_{W3.25} = 19.9$ $U_{W3.25} = 0.05$

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

b. Roof

$$Roof_{3.25} := \begin{pmatrix} 0.1in \\ t_{r.e} \\ 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{pmatrix} \quad Roof_{3.25} = \begin{pmatrix} 0.1 \\ 4 \\ 1 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.3.25} := diag\left(Roof_{3.25} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.3.25} = \begin{pmatrix} 0.25 \\ 1.8 \\ 5 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R3.25} := \sum\left(R_{R.3.25}\right) \qquad U_{R3.25} := \frac{1}{R_{R3.25}}$$

$R_{R3.25} = 21.22 \quad U_{R3.25} = 0.047$

$$Roof_{4.75} := \begin{bmatrix} 0.1in \\ t_{r.e} \\ (2)\cdot 1in \\ 1.5in \\ 0.75in \\ 0.1in \\ 0in \\ 0.5in \end{bmatrix} \quad Roof_{4.75} = \begin{pmatrix} 0.1 \\ 4 \\ 2 \\ 1.5 \\ 0.75 \\ 0.1 \\ 0 \\ 0.5 \end{pmatrix} \cdot in \quad R_{R.4.75} := diag\left(Roof_{4.75} \cdot Comp^{\langle 1 \rangle^T}\right) \quad R_{R.4.75} = \begin{pmatrix} 0.25 \\ 1.8 \\ 10 \\ 9.9 \\ 0.93 \\ 0.68 \\ 0 \\ 2.66 \end{pmatrix}$$

$$R_{R4.75} := \sum\left(R_{R.4.75}\right) \qquad U_{R4.75} := \frac{1}{R_{R4.75}}$$

$R_{R4.75} = 26.22 \quad U_{R4.75} = 0.038$

c. Floor

$W_f = 10.958\,ft$ Width of Floor

$L_f^T = (11.292 \quad 13.292 \quad 15.292 \quad 17.292 \quad 19.292 \quad 21.292 \quad 23.292 \quad 25.292 \quad 27.292)\,ft$ Length of Floor

$wgt_{flr}^T = (41.147 \quad 40.396 \quad 39.841 \quad 39.415 \quad 39.077 \quad 38.803 \quad 38.575 \quad 37.781 \quad 37.662)\cdot psf$

Uniform Distributed Weight of Concrete Floor Only

$$t_{f.conc.avg} := \frac{\max\left(wgt_{flr}\right)}{\gamma_{LW}} \qquad t_{f.conc.avg} = 4.184\cdot in$$

Effective Thickness of Concrete

$$t_{foam.avg} := t_{floor} - t_{f.conc.avg} \qquad t_{foam.avg} = 3.816\cdot in$$

Effective Thickness of Floor Foam Block

$$R_{fl} := \begin{pmatrix} t_{f.conc.avg} \cdot R_{conc} \\ 0.1in\, R_{i.a.f} \\ t_{foam.avg} \cdot \frac{3.85}{in} \end{pmatrix} \quad R_{fl} = \begin{pmatrix} 1.883 \\ 0.68 \\ 14.69 \end{pmatrix} \quad R_F := \sum\left(R_{fl}\right) \quad U_F := \frac{1}{R_F}$$

$R_F = 17.253 \quad U_F = 0.058$

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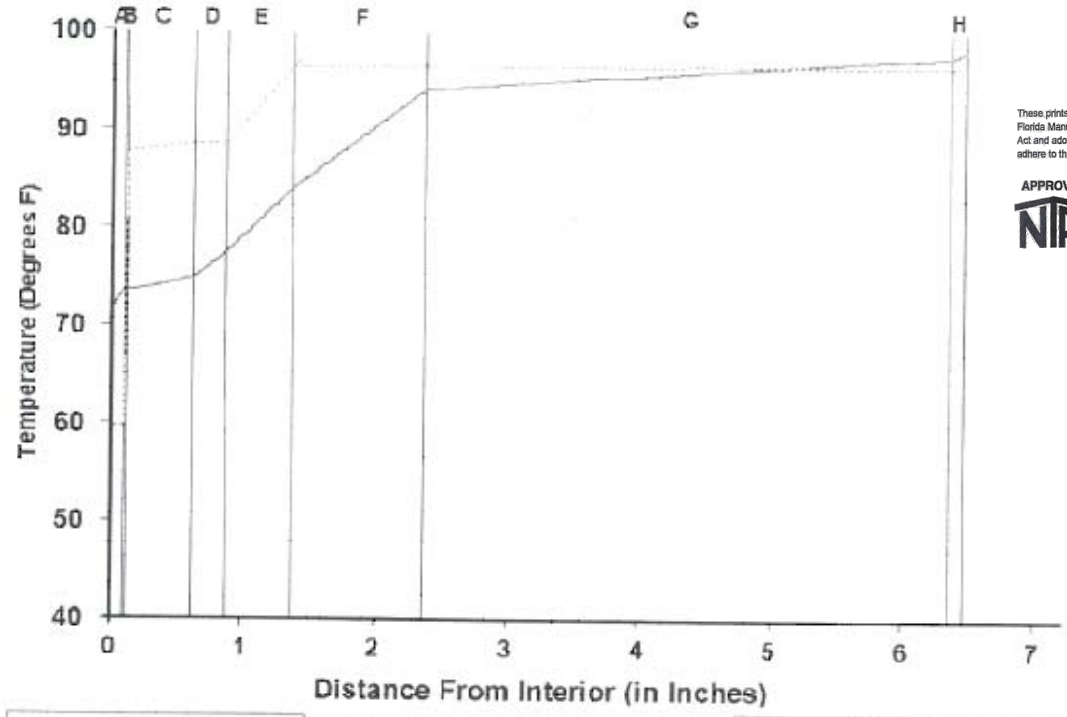
Dewpoint Analysis - Dow Chemical

Sam Cain



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Manufacturer:	Modular Connections, LLC

Legend
Actual Temperature
Dewpoint Temperature

Dewpoint Theory predicts condensation in a system at any point where the actual and dewpoint temperature lines cross

Conditions:	Interior	Exterior
Temperature	72.0	98.0
Humidity	65.0	95.0

	Component Name	Thickness	R-Value	Rep	Interface	Temperature Actual	Temperature Dewpnt	Accum
					-A	72.00	59.55	0.000
A	Interior Air Film	0.100	0.68	0.001	AB	73.36	59.55	0.000
B	Polyethylene 20 mil	0.020	0.01	66.000	BC	73.38	87.56	* 0.001
C	Exterior Plywood .5 in	0.500	0.62	2.800	CD	74.62	88.37	* 0.001
D	Wall Air Space Reflect	0.250	1.33	0.002	DE	77.28	88.37	* 0.001
E	TUFF-R Insulation	0.500	3.30	30.000	EF	83.89	96.05	* 0.022
F	STYROFOAM Scoreboard	1.000	5.00	0.900	FG	93.90	96.26	* 0.015
G	Lightweight Concrete	4.000	1.80	0.440	GH	97.50	96.36	0.000
H	Out Air Film Summer	0.100	0.25	0.001	HI	98.00	96.36	0.000
I					IJ			
J					JK			
K					KL			
L					L-			
	TOTAL	6.470	12.99	100.144				

NOTICE: This calculation is based on the theory of Water Vapor Migration presented in the ASHRAE 1993 Fundamentals Handbook. Actual performance may vary depending upon air infiltration, condensing, and building materials. Since the information is provided without charge, The Dow Chemical Company assumes no obligation or liability for its use.

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10. WALL OPENINGS

a. Opening Sizes

$w_{d.1} := 36in$ Width of Door Opening Opt. 1

$w_{d.2} := 40in$ Width of Door Opening Opt. 2

$w_{d.3} := 48in$ Width of Door Opening Opt. 3

$w_{d.4} := 72in$ Width of Door Opening Opt. 4

$h_d := 84in$ Height of Door Openings

$w_{AC} := 18in$ Width of AC Window Unit

$H_{AC} := 28in$ Height of AC Window Unit

$w_{HVAC.1.2.sup} := 8in$ Width of HVAC Supply Unit Opt. 1, 2

$w_{HVAC.3.sup} := 10in$ Width of HVAC Supply Unit Opt. 3

$h_{HVAC.1} := 20in$ Hgt of HVAC Supply/Return Unit Opt. 1

$h_{HVAC.2} := 28in$ Hgt of HVAC Supply/Return Unit Opt. 2

$h_{HVAC.3} := 30in$ Hgt of HVAC Supply/Return Unit Opt. 3

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Manufacturer:	Modular Connections, LLC

$w_{HVAC.1.rtn} := 12in$ Width of HVAC Return Unit Opt. 1

$w_{HVAC.2.rtn} := 14in$ Width of HVAC Return Unit Opt. 2

$w_{HVAC.3.rtn} := 16in$ Width of HVAC Return Unit Opt. 3

$d_{HVAC.1} := 20in$ Below Opening HVAC Unit Opt. 1

$d_{HVAC.2} := 18in$ Below Opening HVAC Unit Opt. 2

$d_{HVAC.3} := 30in$ Below Opening HVAC Unit Opt. 3

$w_{wave} := 20.75in$ Width of Wave Guide Plates

$h_{wave.1} := 17in$ Hgt of Wave Guide Opt. 1, 4 x 12

$h_{wave.2} := 11in$ Hgt of Wave Guide Opt. 1, 4 x 8

$h_{wave.3} := 4in$ Hgt of Wave Guide Opt. 1, 4 x 4

$w_{exhaust.1} := 12.5in$ Width & Height of 12" Sq Exhaust

$w_{exhaust.2} := 16.5in$ Width & Height of 16" Sq Exhaust

$w_{exhaust.3} := 18.5in$ Width & Height of 18" Sq Exhaust

$w_{exhaust.4} := 20.5in$ Width & Height of 20" Sq Exhaust

$w_{louver} := 42.5in$ Width of Louver

$h_{louver} := 50.5in$ Height of Louver

Opening Combinations:

- -Door and AC
- -(2) HVAC and Wave Guide
- -Door and Exhaust
- -Maximum Allowable Block Out Available

b. Out of Plane Design for End Walls

$w := \max(|P_{W.c}|, |P_{L.c}|, |P_{LC.c}|)$ $w = 79.561 \cdot psf$ Maximum Wind Wall Pressure

$M_{max} := \max\left(\frac{w \cdot H_{wall}^2 \cdot 12in}{8}\right)$ $M_{max} = 1131.5 \cdot lb \cdot ft$ Maximum Moment per foot due to Out of Plane Wind Pressure

Flexure: Door Size = 6'-0" $LF_{wind} := 1.0$

$M_{6.d} := LF_{wind} \cdot M_{max} \cdot \frac{W_{ew} - 4in}{(W_{ew} - 4in) - w_{d.4}}$ $M_{6.d} = 2405 \cdot lb \cdot ft$ Maximum Moment per foot On Wall minus Wall Opening

$d := 0.5t...$ $d = 2 \cdot in$ Depth of Reinf.

$$f(x) := \frac{\frac{M_{6.d}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left(\frac{d}{in}\right)^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.01112$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.01252$ Reinf. Ratio Req'd

$A_{s.rq} := \rho \cdot \max(12in) \cdot d$ $A_{s.rq} = 0.3 \cdot in^2$ Req'd Area of Steel per foot Width

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Occupancy: S2
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Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

Shear:

$\phi V_c := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot d$ $\quad \phi V_c = 1745 \cdot lb$ Factored Shear Strength of Wall

$V_{u.o.plane} := \max\left(LF_{wind} \cdot w \cdot \frac{H_{wall}}{2}\right) \cdot 12in$ $\quad V_{u.o.plane} = 424 \cdot lb$ Factored Shear Force per Foot Width

$chk_{V.d.opn} := if(\phi V_c \geq 2 \cdot V_{u.o.plane}, "OK", "NG")$ $\quad chk_{V.d.opn} = "OK"$

With Penetrations

$\rho_{bal} := 0.75 \cdot \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000psi}{87000psi + f_y}\right)$ $\quad \rho_{bal} = 0.02335$ $\quad A_s := \rho_{bal} \cdot (d \cdot 12in)$ $\quad A_s = 0.56 \cdot in^2$

Use ($n_{bars} := 2$) $D_{bar.11.1} := "\#5"$ Bars around openings

$D_{11.1} = 0.625 \cdot in$ $\quad A_{11.1} = 0.31 \cdot in^2$

$a := \frac{n_{bars} \cdot A_{11.1} \cdot f_y}{0.85 \cdot f'_c \cdot 12in}$ $\quad a = 0.81 \cdot in$ $\quad M_{allow} := 0.9 \cdot n_{bars} \cdot A_{11.1} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_{allow} = 4.449 \cdot kip \cdot ft$

$V_{allow} := \phi_V \cdot 0.85 \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot c$ $\quad V_{allow} = 1745 \cdot lb$ $\quad w_{allow} := \min\left(\frac{8 \cdot M_{allow}}{H_{wall}^2}, \frac{2V_{allow}}{H_{wall}}\right)$ $\quad w_{allow} = 312.849 \cdot plf$

$w_{d.strip} := 2 \cdot \left(\frac{w_{allow}}{w} - 12in\right)$ $\quad w_{d.strip} = 5.864\,ft$ $\quad w_d := 2 \cdot w_{d.strip}$ $\quad w_d = 11.729\,ft$

Maximum allowable opening width

$w_{d.max} := \max(w_{d.1}, w_{d.2}, w_{d.3}, w_{d.4})$ $\quad w_{d.max} = 6\,ft$ Maximum Door width

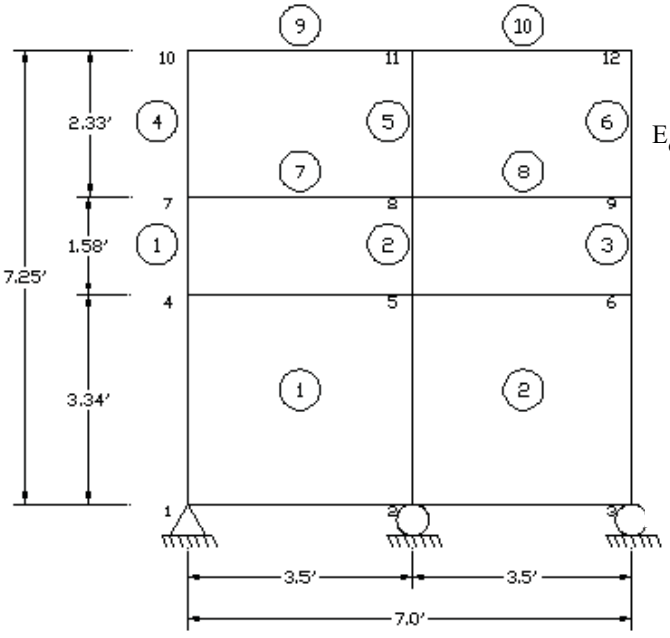
$chk_{opn.width} := if(w_d \geq w_{d.max}, "OK", "NG")$ $\quad chk_{opn.width} = "OK"$

$V_{u.EW.0} = 4.456 \cdot kip$ $\quad V_{u.EW.0.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 9\ '\ -\ 0\ "\ "$

$V_{u.EW.1} = 4.456 \cdot kip$ $\quad V_{u.EW.1.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 0\ "\ "$

$V_{u.EW.2} = 4.456 \cdot kip$ $\quad V_{u.EW.2.ref} = "Max\ Shear\ Force\ @\ Top\ of\ Endwall,\ for\ Wall\ Height\ 10\ '\ -\ 8\ "\ "$

$V_u := \max(V_{u.EW.0}, V_{u.EW.1}, V_{u.EW.2})$ In-Plane Force at Endwall



← $V_u = 4.456 \cdot kip$

$E_{opn} :=$

Member	d (in)	A (in²)	I (in⁴)	V_u (kip)	M_u (kip-ft)
1	10	40	333	-2.01	3.18
2	10	40	333	-3.12	4.92
3	10	40	333	-1.92	3.03
4	10	40	333	-1.68	2.34
5	10	40	333	-3.24	4.08
6	10	40	333	-2.11	2.78
7	12	48	576	2.52	4.76
8	12	48	576	2.72	4.33
9	12	48	576	1.22	2.34
10	12	48	576	1.41	2.78

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$d_1 = 10\cdot in$	$Ag_1 = 40\cdot in^2$	$Ig_1 = 333.333\cdot in^3$	$Vu_1 = 2.01\cdot kip$	$Mu_1 = 3.18\cdot kip\cdot ft$
$d_2 = 10\cdot in$	$Ag_2 = 40\cdot in^2$	$Ig_2 = 333.333\cdot in^3$	$Vu_2 = 3.12\cdot kip$	$Mu_2 = 4.92\cdot kip\cdot ft$
$d_3 = 10\cdot in$	$Ag_3 = 40\cdot in^2$	$Ig_3 = 333.333\cdot in^3$	$Vu_3 = 1.92\cdot kip$	$Mu_3 = 3.03\cdot kip\cdot ft$
$d_4 = 10\cdot in$	$Ag_4 = 40\cdot in^2$	$Ig_4 = 333.333\cdot in^3$	$Vu_4 = 1.68\cdot kip$	$Mu_4 = 2.34\cdot kip\cdot ft$
$d_5 = 10\cdot in$	$Ag_5 = 40\cdot in^2$	$Ig_5 = 333.333\cdot in^3$	$Vu_5 = 3.24\cdot kip$	$Mu_5 = 4.08\cdot kip\cdot ft$
$d_6 = 10\cdot in$	$Ag_6 = 40\cdot in^2$	$Ig_6 = 333.333\cdot in^3$	$Vu_6 = 2.11\cdot kip$	$Mu_6 = 2.78\cdot kip\cdot ft$

Section with 10" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.10.opn} := \max(Vu_1, Vu_2, Vu_3, Vu_4, Vu_5, Vu_6)$ $V_{u.10.opn} = 3.24\cdot kip$

$M_{u.10.opn} := \max(Mu_1, Mu_2, Mu_3, Mu_4, Mu_5, Mu_6)$ $M_{u.10.opn} = 4.92\cdot kip\cdot ft$

Shear

$\phi Vc_{10.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_1 - 1.5in)$ $\phi Vc_{10.opn} = 2.908\cdot kip$

$\phi Vs_{10.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_1 - 1.5in)}{4in}$ $\phi Vs_{10.opn} = 11.475\cdot kip$

$\phi Vn_{10.opn} := \phi Vc_{10.opn} + \phi Vs_{10.opn}$ $\phi Vn_{10.opn} = 14.383\cdot kip$

$chk_{V.10.opn} := if(\phi Vn_{10.opn} \geq 2\cdot V_{u.10.opn}, "OK", "NG")$ $chk_{V.10.opn} = "OK"$

Flexure

$$f(x) := \frac{\dfrac{M_{u.10.opn}}{kip\cdot ft}}{\phi_{b.TC}\cdot\dfrac{f_y}{ksi}\cdot\dfrac{12in}{ft}\cdot\left[\dfrac{(d_1 - 1.5in)}{in}\right]^2\cdot(1 - 8.85\cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.02$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00128$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.10.opn} := \rho\cdot\max(12in)\cdot(d_1 - 1.5in)$ $A_{s.rq.10.opn} = 0.13\cdot in^2$ Req'd Area of Steel Adjacent to Opening

$d_7 = 12\cdot in$	$Ag_7 = 48\cdot in^2$	$Ig_7 = 576\cdot in^3$	$Vu_7 = 2.52\cdot kip$	$Mu_7 = 4.76\cdot kip\cdot ft$
$d_8 = 12\cdot in$	$Ag_8 = 48\cdot in^2$	$Ig_8 = 576\cdot in^3$	$Vu_8 = 2.72\cdot kip$	$Mu_8 = 4.33\cdot kip\cdot ft$
$d_9 = 12\cdot in$	$Ag_9 = 48\cdot in^2$	$Ig_9 = 576\cdot in^3$	$Vu_9 = 1.22\cdot kip$	$Mu_9 = 2.34\cdot kip\cdot ft$
$d_{10} = 12\cdot in$	$Ag_{10} = 48\cdot in^2$	$Ig_{10} = 576\cdot in^3$	$Vu_{10} = 1.41\cdot kip$	$Mu_{10} = 2.78\cdot kip\cdot ft$

Section with 12" Wide Wall Between Openings Worst Shear and Moment Considered

$V_{u.12.opn} := \max(Vu_7, Vu_8, Vu_9, Vu_{10})$ $V_{u.12.opn} = 2.72\cdot kip$

$M_{u.12.opn} := \max(Mu_7, Mu_8, Mu_9, Mu_{10})$ $M_{u.12.opn} = 4.76\cdot kip\cdot ft$

Shear

$\phi Vc_{12.opn} := \phi_V\cdot 0.85\cdot 2\cdot\sqrt{f'_c\cdot psi}\cdot t_w\cdot(d_7 - 1.5in)$ $\phi Vc_{12.opn} = 3.592\cdot kip$

$\phi Vs_{12.opn} := \dfrac{\phi_V\cdot(WWF_{4x4\_w4xw4}\cdot 1ft)\cdot f_y\cdot(d_7 - 1.5in)}{4in}$ $\phi Vs_{12.opn} = 14.175\cdot kip$

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$\phi Vn_{12.opn} := \phi Vc_{12.opn} + \phi Vs_{12.opn}$ $\phi Vn_{12.opn} = 17.767 \cdot kip$

$chk_{V.12.opn} := if\left(\phi Vn_{12.opn} \geq 2 \cdot V_{u.12.opn}, "OK", "NG"\right)$ $chk_{V.12.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.12.opn}}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00081$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00081$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.12.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.12.opn} = 0.102 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

Section with 6' Wide Door Opening

$V_{u.6d.opn} := V_u$ $V_{u.6d.opn} = 4.456 \cdot kip$

$M_{u.6d.opn} := V_u \cdot \frac{w_{d.4}}{2}$ $M_{u.6d.opn} = 13.367 \cdot kip \cdot ft$

Shear

$\phi Vc_{.6d.opn} := \phi_{.V} \cdot 0.85 \cdot 2 \cdot \sqrt{f'_{.c} \cdot psi} \cdot t_{.w} \cdot \left(d_{.7} - 1.5in\right)$ $\phi Vc_{6d.opn} = 3.592 \cdot kip$

$\phi Vs_{6d.opn} := \frac{\phi_V \cdot \left(WWF_{4x4\_w4xw4} \cdot 1ft\right) \cdot f_y \cdot \left(d_7 - 1.5in\right)}{4in}$ $\phi Vs_{6d.opn} = 14.175 \cdot kip$

$\phi Vn_{6d.opn} := \phi Vc_{6d.opn} + \phi Vs_{6d.opn}$ $\phi Vn_{6d.opn} = 17.767 \cdot kip$

$chk_{V.dr.opn} := if\left(2 \cdot \phi Vn_{6d.opn} \geq 2 \cdot V_{u.6d.opn}, "OK", "NG"\right)$ $chk_{V.dr.opn} = "OK"$

Flexure

$$f(x) := \frac{\frac{M_{u.6d.opn} \cdot 0.5}{kip \cdot ft}}{\phi_{b.TC} \cdot \frac{f_y}{ksi} \cdot \frac{12in}{ft} \cdot \left[\frac{\left(d_7 - 1.5in\right)}{in}\right]^2 \cdot (1 - 8.85 \cdot x)} - x$$

Formula for Req'd Reinf. Ratio

$x := 0.00252$ Initial Guess $\rho := root(f(x), x)$ $\rho = 0.00113$ Reinf. Ratio Req'd Adjacent to Opening

$A_{s.rq.6d.opn} := \rho \cdot max(12in) \cdot \left(d_7 - 1.5in\right)$ $A_{s.rq.6d.opn} = 0.143 \cdot in^2$ Req'd Area of Steel Adjacent to Opening

$At_{opn.p} := 2 \cdot A_5$ $At_{opn.p} = 0.62 \cdot in^2$ Area of Steel Provided around openings (2 layers)

$chk_{M.opn} := if\left(At_{opn.p} \geq A_{s.rq}, "OK", "NG"\right)$ $chk_{M.opn} = "OK"$

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

MODULAR CONNECTIONS, LLC.
1090 Industrial Blvd.
Bessemer, AL 35022
Ph# 205-980-4565 Fx# 877-675-5851
Email: info@modularconnections.com

APPENDIX "A"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MFT-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

PFS Corporation
IN SOUTH WESTGATE DRIVE
GREENSBORO, NORTH CAROLINA 27407 TELEPHONE (919) 854-1844

PROJECT: Bullet Resistance Test

DATE: February 6, 1990

Reported To: UNR-Rohn
P.O. Box 2000
Peoria, Illinois 62656

On January 16, 1990, our representative, Melvin Johnson arrived at Van Doren Industries, Hays, Kansas and was met by Mr. Richard Malmberg, General Manager. Mr. Malmberg and our representative discussed the bullet test procedures to be followed before conducting the actual tests. The bullet resistance test was to be conducted on a wall assembly of the Standard Rohn Concrete Shelter.

The tests were designed as closely as possible to meet U.L. 752 weapon and ammunition specifications. The test procedure followed the U.L. listed specifications as shown below.

1) Range - 15 feet or less

2) Sample - Rigidly mounted with corrugated cardboard 18" behind

3) High Powered Rifle - Based on Sample resistance to one shot.

4) Tests Conducted - At room temperature, 22 degrees + 3 degrees C. (71.6 degrees + 5 degrees F). There must be no penetration of projectile through test sample, and no spalling of material on projected site to extent that fragments inbed in or damage cardboard.

Our representative witnessed the test as follows:

Test Panel - Side panel of subject enclosure

Material of Panel - 4" thick concrete - 112 density - 4000 psi composed of expanded shale light weight coarse aggregate with regular weight sand finer back surface of panel - 1" foam board and 1/2" plywood fastened to panel.

Bullet Test Weapon/Ammunition - (1) high power rifle 24" barrel (2) Remington bullets -30-06 rifle-high velocity - 220 grain soft point core Lokt-R 30067.

Test Sample - Material was of commercial construction as noted above. The six test shots were fired from a distance of 15 feet and in a grid pattern with cardboard indicator, approximately 1/8" thick, double face, was placed at a distance of 18 inches behind the protected side of the test panel.

Observation - Using the designated rifle and ammunition, shots were placed at various locations on the test panel to allow penetration of the projectile with sufficient force to inbed into the panel. It was found in all six samples, that the projectile was imbedded into the panel approximately 1 1/2 inches. There was no penetration of the wall section by the projectile or of any other material. There was no debris released into the interior as the result of the impact and there were

no visible signs on the interior surface of the projectiles' impact.

Conclusion - The above referenced, as modified, Bullet Resistance Test was conducted using UL 752 as the criteria. None of the six (6) samples witnessed allowed the bullet allowed the bullet to penetrate through the test specimen. The samples prevented any debris or damage as required.

Thank You.

Sincerely,

Ken W. Houglan
General Manager, Southeast Region

cc: Rick Malmberg
Melvin Johnson
Larry Beineke

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Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
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APPENDIX "B"

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Ult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

Awning / Hood Anchor Design

Design Criteria

Models, Dimensions & Wind Load

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	150	mph
Struc Category (I, II, III, or	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.0	:12
Slope of Roof (Theta)	0.0	Deg
Type of Roof	Gabled	
Kd (Directonality Factor)	0.85	
Eave Height (Eht)	10.00	ft
Ridge Height (RHt)	10.00	ft
Mean Roof Height (Ht)	10.00	ft
Width Perp. To Wind Dir (E	36.00	ft
Width Paral. To Wind Dir (	12.00	ft

Calculated Parameters		
Importance Factor	1	
Hurricane Prone Region (V>100 mph)		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.83
Flexible Structure	No

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
lzm	Cc * (33/z)^0.167	0.2281	
Lzm	l*(zm/33)^Epsilon	427.06	ft
Q	(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^0.63))^0.5	0.9547	
Gust2	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	0.9011	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	**0.18**	**-0.18**

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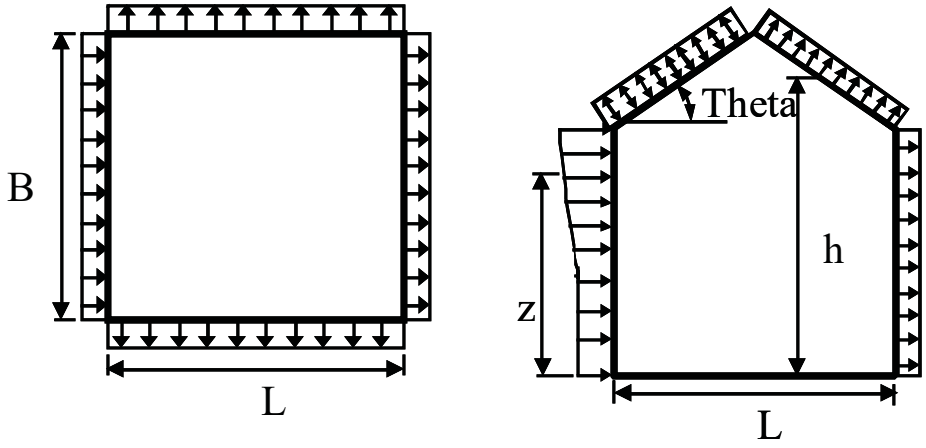
6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev	Kz	Kzt	qz	Pressure (lb/ft^2)	
				Windward Wall*	
0			lb/ft^2	+GCpi	-GCpi
15	#NAME?	1.00	#NAME?	#NAME?	#NAME?

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	2.01*(15/zg)^(2/Alpha)	0.85	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	.00256*(V)^2*I*Kh*Kht*Kd	41.56	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.85	
Qhcc	.00256*V^2*I*Khcc*Kht*Kd	41.56	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00

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Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MFT-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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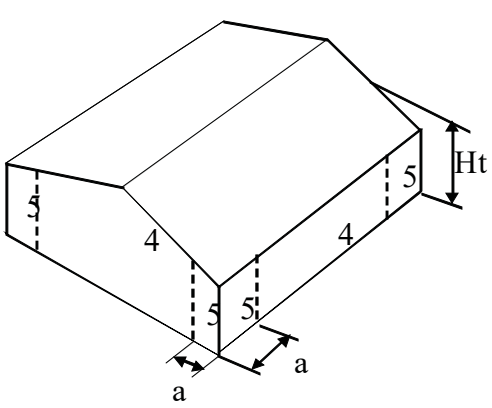
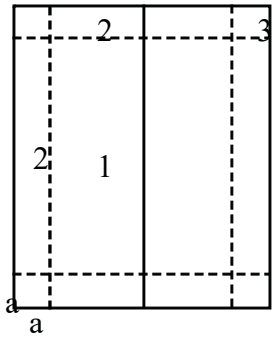
Calculations for Wind Normal to 36 ft Face *Additional Runs may be req'd for other wind directions*	Cp	Pressure (psf) +GCpi	 -GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.50	-25.14	-10.18
Leeward Walls (Wind Dir Normal to 12 ft wall)	-0.25	-16.31	-1.35
Side Walls	-0.70	-32.21	-17.25
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-1.17	-48.69	-33.73
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.77	-34.56	-19.60
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.63	-29.85	-14.89
Dist from Windward Edge: > 20 ft	-1.30	-53.40	-38.44
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 12 ft face			
Dist from Windward Edge: 0 ft to 20 ft - Max Cp	-0.18	-13.84	1.12
Dist from Windward Edge: 0 ft to 5 ft - Min Cp	-0.72	-32.99	-18.03
Dist from Windward Edge: 5 ft to 10 ft - Min Cp	-0.90	-39.27	-24.31
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.50	-25.14	-10.18
Dist from Windward Edge: > 20 ft	-0.30	-18.08	-3.12

* Horizontal distance from windward edge

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht <= 60 ft





Gabled Roof
Theta <= 7

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Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
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Email: info@modularconnections.com

a = 1.2 ==> 3.00 ft

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft^2)	Zone	GCp		Wind Press (lb/ft^2)	
					Max	Min	Max	Min
Fiberglass Awning 36x56	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES38354	2	9	27.00	3H	0.26	-1.94	10.81	-80.63
ES4471A	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES11778	2	7	16.33	3H	0.28	-2.37	11.64	-98.50
ES4471SS	3	4.667	14.00	3H	0.29	-2.51	12.05	-104.32
ES96SS	1.708	1.708	2.92	3H	0.30	-2.80	12.47	-116.37
ES12075SS	3.375	3.375	11.39	3H	0.29	-2.69	12.05	-111.80
ES4379SS	4.375	4.375	19.14	3H	0.27	-2.24	11.22	-93.09

Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) 116.5 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
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Manufacturer:	Modular Connections, LLC

MC P/N	Width (in)	Depth (in)	Height (in)	Material	Wind Pressure (lb/ft^2)	Model Label for Calcs
Fiberglass 36x56	56	36	8	1/8" Fiberglass	104.22	A
ES38354	108	24	8	1/8" Aluminum	80.52	B
ES4471A	56	36	8	1/8" Aluminum	104.22	C
ES11778	84	24	8	1/8" Aluminum	98.66	D
ES4471SS	56	36	8	16 GA 304SS	104.22	E
ES96SS	20.5	20.5	20.5	17 GA 304SS	116.37	F
ES12075SS	40.5	40.5	40.5	18 GA 304SS	111.67	G
ES4379SS	52.5	52.5	52.5	19 GA 304SS	92.94	H

Minimum Total Number of Anchors

The quantity of anchors listed below are for shear requirements only. The pullout force generated is accounted for in the 2 anchors attached to the angle just above and below the bottom edge of the awning / hood.

$No_A := 9$ # of Anchors Model "A"

$No_B := 12$ # of Anchors Model "B"

$No_C := 9$ # of Anchors Model "C"

$No_D := 10$ # of Anchors Model "D"

$No_E := 9$ # of Anchors Model "E"

$No_F := 16$ # of Anchors Model "F"

$No_G := 18$ # of Anchors Model "G"

$No_H := 20$ # of Anchors Model "H"

Anchor Loads

$V_{A\_ea} := \frac{V_A}{No_A} = 162.12 \cdot lb$ Total Shear per Anchor *Model "A"*

$T_A := \frac{w_A \cdot W_A \cdot D_A^2}{4H_A} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "A"*

$V_{B\_ea} := \frac{V_B}{No_B} = 120.78 \cdot lb$ Total Shear per Anchor *Model "B"*

$T_B := \frac{w_B \cdot W_B \cdot D_B^2}{4H_B} = 1087 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "B"*

$V_{C\_ea} := \frac{V_C}{No_C} = 162.12 \cdot lb$ Total Shear per Anchor *Model "C"*

$T_C := \frac{w_C \cdot W_C \cdot D_C^2}{4H_C} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "C"*

$V_{D\_ea} := \frac{V_D}{No_D} = 138.124 \cdot lb$ Total Shear per Anchor *Model "D"*

$T_D := \frac{w_D \cdot W_D \cdot D_D^2}{4H_D} = 1035.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "D"*

$V_{E\_ea} := \frac{V_E}{No_E} = 162.12 \cdot lb$ Total Shear per Anchor *Model "E"*

$T_E := \frac{w_E \cdot W_E \cdot D_E^2}{4H_E} = 1641.5 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "E"*

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$V_{F\_ea} := \frac{V_F}{No_F} = 21.226 \cdot lb$ Total Shear per Anchor *Model "F"*

$T_F := \frac{w_F \cdot W_F \cdot D_F^2}{4H_F} = 84.9 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "F"*

$V_{G\_ea} := \frac{V_G}{No_G} = 70.666 \cdot lb$ Total Shear per Anchor *Model "G"*

$T_G := \frac{w_G \cdot W_G \cdot D_G^2}{4H_G} = 318 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "G"*

$V_{H\_ea} := \frac{V_H}{No_H} = 88.9 \cdot lb$ Total Shear per Anchor *Model "H"*

$T_H := \frac{w_H \cdot W_H \cdot D_H^2}{4H_H} = 444.7 \cdot lb$ Tension Load on Single Bottom Anchor (Pull Out) *Model "H"*

Anchor Allowable Loads

$V_{a\_Tap} := 320lb$ Total Shear per Anchor 3/16"x 1-1/2"Embed Titan Masonry Screw Minimum Edge Distance 2¼"

$T_{a\_Tap} := 760lb$ Tension Load on Single Bottom Anchor (Pull Out) 3/16"x 1-1/2" Embed Titan Masonry Screw Minimum Edge Distance 2¼"

Shear Check

$V_{max} := \max(V_{A\_ea}, V_{B\_ea}, V_{C\_ea}, V_{D\_ea}, V_{E\_ea}, V_{F\_ea}, V_{G\_ea}, V_{H\_ea}) = 162.12 \cdot lb$ $V_{a\_Tap} = 320 \cdot lb$

$Shear_{check} := \text{if}(V_{a\_Tap} \geq V_{max}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ Shear Requirements are OK

Angle (Material : Aluminum)

$F_y := 35ksi$ $F_u := 38ksi$ $G := 39000ksi$ Angle L 2 x 2 x 5/16 $b := 2in$ $t := \frac{5}{16}in$ $S := 0.298in^3$

$Z := 0.536in^3$ $L_A := 22in$ $L_B := 58in$ $S_c := 0.8 \cdot S = 0.238 \cdot in^3$ $E_{al} := 10000ksi$ $C_b := 1.5$

$M_y := F_y \cdot Z$ $M_{n\_yield} := 1.5 \cdot M_y = 28.14 \cdot kip \cdot in$

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Moment Capacity of Angle for Awingings

$$M_{e\_A} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_A^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_A \cdot t}{b^2} \right)^2} - 1 \right] = 83.633 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_A}}{M_y} \right) \cdot M_{e\_A} = 13.6 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_A := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 13.6 \cdot kip \cdot in$$

Moment Capacity of Angle for Hoods

$$M_{e\_B} := \frac{0.66 \cdot E_{al} \cdot b^4 \cdot t \cdot C_b}{L_B^2} \cdot \left[\sqrt{1 + 0.78 \cdot \left(\frac{L_B \cdot t}{b^2} \right)^2} - 1 \right] = 45.982 \cdot kip \cdot in \quad M_{n\_LTB} := \left(0.92 - 0.17 \cdot \frac{M_{e\_B}}{M_y} \right) \cdot M_{e\_B} = 23.1 \cdot kip \cdot in$$

$$M_{n\_LLB} := F_y \cdot S_c \cdot \left(2.43 - 1.72 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E_{al}}} \right) = 14.842 \cdot kip \cdot in \qquad M_B := \min\left(M_{n\_yield}, M_{n\_LTB}, M_{n\_LLB} \right) = 14.8 \cdot kip \cdot in$$

$M_{a\_A} := 1641.5lb \cdot 8in = 13.1 \cdot kip \cdot in$ Max Bending Moment for Awnings

$M_{a\_B} := 444.7lb \cdot 20.5in = 9.1 \cdot kip \cdot in$ Max Bending Moment for Hoods

$Bending_{check} := if\left(M_A \geq M_{a\_A} \wedge M_B \geq M_{a\_B}, "OK", "NG" \right) = "OK"$ Angle Flexural Requirements are OK

Distribution of Pullout to Anchors

$T_a := 760lb$

Tension Load on Single
Bottom Anchor (Pull Out)
3/16"x 1-1/2" Embed Titan Masonry Screw
Minimum Edge Distance 2¼"

$WF := 1.33$ Wind Stress Reduction Factor

$$T_{max\_A} := \frac{\max\left(T_A, T_B, T_C, T_D, T_E \right)}{2WF} = 617.092 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Awning.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

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$$T_{max\_B} := \frac{\max(T_F, T_G, T_H)}{2WF} = 167.193 \cdot lb$$

Max Pullout per anchor 2 1/4" either side of Bottom Edge of Hood.
These 2 anchors carry no shear, shear is carried by the quantity of anchors listed above.

$$Pullout_{check} := if(T_a \geq T_{max\_A} \wedge T_a \geq T_{max\_B}, "OK", "NG") = "OK"$$ Pullout Requirements are OK

Use L 2 x 2 x 5/16 Aluminum Angle
Length of:
22" for all Awnings
58" for all Hoods

Use 3/16"x 1-1/2"Embed Titan Masonry Screw 4 1/2" O.C. with one 2 1/4" above bottom edge of Awning or Hood and one 2 1/4" below bottom edge of Awning or Hood with 4 1/2" CTC spacing in the remainder of the Angle.

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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APPENDIX "C"

These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type:	VB
Occupancy:	S2
Allowable No. of Floors:	One (1)
Wind Velocity:	150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls:	2 HR
Plan No.:	MET-4193-Standard Concrete
Allow. Floor Load:	150 PSF(200 PSF & 250 PDF
Approval Date:	5/30/2024
Manufacturer:	Modular Connections, LLC

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These prints comply with the Florida Manufactured Building Act and adopted Codes and adhere to the following criteria:

APPROVED BY NTA INC.

Const. Type: VB
Occupancy: S2
Allowable No. of Floors: One (1)
Wind Velocity: 150 MPH (Vult) [165 MPH (V
Fire Rating of Ext. Walls: 2 HR
Plan No.: MET-4193-Standard Concrete
Allow. Floor Load: 150 PSF(200 PSF & 250 PDF
Approval Date: 5/30/2024
Manufacturer: Modular Connections, LLC

DEVELOPMENT OF REINFORCEMENT IN TENSION

Code: ACI 318-05 (Section 12.2)

Basic Development Length:

$$l_{db} = \frac{f_y * d_b}{25*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #6 bars and smaller)

$$l_{db} = \frac{f_y * d_b}{20*(f'_c)^{0.5}} \quad {}_t \; {}_e$$ (for #7 bars and larger)

Material Properties:

f_y =	60000	ksi
f'_c =	4000	ksi
$_e$ =	1.2	(Use factors from below)
$_t$ =	1.0	(Use factors from below)
0=	1.0	(Use factors from below)

Bar Size	$_b$	l_{db}	$_t$	$_e$		l_d
#3	0.375	14.2	1.0	1.2	1.0	17.1
#4	0.5	19.0	1.0	1.2	1.0	22.8
#5	0.625	23.7	1.0	1.2	1.0	28.5
#6	0.75	28.5	1.0	1.2	1.0	34.2
#7	0.875	41.5	1.0	1.2	1.0	49.8
#8	1	47.4	1.0	1.2	1.0	56.9
#9	1.125	53.4	1.0	1.2	1.0	64.0
#10	1.25	59.3	1.0	1.2	1.0	71.2
#11	1.375	65.2	1.0	1.2	1.0	78.3

Modification Factors:

$_t$ = Reinforcement Location Factor

=	1.3	Horizontal reinforcement w/more than 12" of fresh concrete cast below splice
=	1.0	all other cases

$_e$ = Coating Factor

=	1.5	epoxy-coated w/cover < $3d_b$ or clear spacing less than $6d_b$
=	1.2	all other epoxy-coated bars
=	1.0	uncoated reinforcement

Excess Reinf. = Reduction in l_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for fy is specifically required or the reinforcement is designed under provisions of 21.2.1.4.

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