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Structural Design Calculations

State of Florida
Department of Transportation
Suwannee County & Columbia County, Florida

GAI Project Number: B190988.00

November 2020

Prepared by: GAI Consultants, Inc.
Cranberry Office
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Cranberry Township, Pennsylvania 16066

Prepared for: State of Florida
Department of Transportation

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Table of Contents

1.0 Project Summary

This report summarizes GAI Consultants, Inc.'s (GAI) structural calculations for the design of the I-10 Rest Areas in Columbia and Suwannee Counties in Florida. The project includes calculations for the main Rest Area, the Picnic Pavilions, the Storage Building and the Dumpster Pavilion. Refer to the GAI structural drawings (Drawing No. B190988 Sheets S-001 thru S-605).

1.1 Purpose

The calculations contained herein address the analysis and design of structural systems required for the Rest Area Structure, the Picnic Pavilions, the Storage Building and the Dumpster Pavilion. The following design aspects were considered:

- Design of Rest Area Structure including Roof, Wall, Columns, and Foundations.
- Design of Picnic Pavilion Roof, Columns, and Foundations.
- Design of Storage Building Roof, Walls, and Foundations.
- Design of Dumpster Pavilion Walls, and Foundations.

1.2 Approach

The following steps are used to solve the issues outlined above:

- A design basis was developed to define the design criteria and parameters. The design basis includes the following:
  - Applicable design codes
  - A summary of the loads (equipment/dead load, seismic, wind, snow)
  - Material properties for the structural members
- The design calculations were performed using the criteria and parameters defined in the design basis. The methods of analysis for the calculations include hand calculations and utilizing the computer programs TEDDS, MathCAD, RISA, HILTI, SPMats and STAAD.

1.3 Analysis Summary

The Buildings and Structures were designed per the Architect design drawings and applicable design loads.

2.0 References

2017 Florida Building Code*q*
q q q , Building, 6th Edition 2017

2015 International Building Code IBC 2015

American Society of Civil Engineers ASCE/SEI 7-10
q q q q q q q q .

American Institute of Steel Construction
q q q q .

American Concrete Institute 318-14
q q q q q q .

ATTACHMENT A
Design Basis

Florida Department of Transportation

Suwannee and Columbia County, FLA.
CONTRACT B190988.00

STRUCTURAL DESIGN BASIS
Rev. A – 4/28/20

1. CODES
    a. Florida Building Code (FBC), 6th Edition 2017
    b. IBC 2015 International Building Code
    c. ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures
    d. ASCE/SEI 6-13, Specification for Masonry Structures
    e. AISC Manual of Steel Construction 14th Edition, ASD
    f. American Welding Society, AWS, D1.1/D1.1M: 2010 Structural Welding Code Steel
    g. ACI 318-14, Building Code Requirements for Reinforced Concrete
    h. ACI 360-10, Guide to Design of Slabs-on Ground
    i. ACI 530-13 Building Code Requirements for Masonry Structures
    j. ANSI/AWC NDS-2015 National Design Specification (NDS) for Wood Construction with 2015 NDS Supplement
    k. AISI S202-15, Code of Standard Practice for Cold-formed Steel Framing

2. Risk Category
    a. Structure Risk Category II, FBC Table 1604.5

3. Dead Loads
    a. Roof: 20psf

4. Live Loads
    a. Roof: 20psf

5. Snow Loads
    a. Project location:
        i. Latitude: 30.27435833
        ii. Longitude: -82.79940833
    b. Snow loading: 5psf

6. Wind Loads
    a. Project location:
        i. Latitude: 30.27435833
        ii. Longitude: -82.79940833

b. Ultimate design wind Speed: 120mph
c. Exposure C, FBC Section 1609.4.3
d. Importance Factor – 1.00 ASCE Table 1.5-2
e. Building Enclosure – Enclosed
f. Internal Pressure Coefficient: see wind calcs

7. Foundation Guidelines
   a. Allowable Bearing Pressure, 2000 psf
   b. Frost line, 6” below grade
   c. Subgrade modulus, 100 pci

8. Seismic Loads
   a. Project location:
      i. Latitude: 30.27435833
      ii. Longitude: -82.79940833
   b. Structure Risk Category = II
   c. Site Class = D (default)
   d. Spectral Accelerations based on site location
      i. $S_S = 0.086$ $S_1 = 0.051$
   e. Site Coefficients (Table 1613.3.3(1) and 1613.3.3(2))
      i. $F_a = 1.6$ $F_v = 2.4$
   f. Maximum considered earthquake spectral response
      i. $S_{MS} = F_aS_s = 0.137$
      ii. $S_{M1} = F_vS_1 = 0.123$
   g. Design spectral response acceleration parameter
      i. $S_{DS} = (2/3)S_{MS} = 0.092$ $S_{D1} = (2/3)S_{M1} = 0.082$
   h. Governing Seismic Design Category __B

9. Structural Material Properties
   a. Structural Steel
      i. Wide Flange shapes: A992, Gr. 50 (Fy = 50 ksi)
      ii. Channels, Misc. channels, Angles, M and S Shapes: A36 (Fy = 36 ksi)
      iii. HSS rectangular: A500, Gr. B (Fy = 46 ksi)
      iv. HSS round: A500, Gr. B (Fy = 42 ksi)
      v. Pipe: A53, Gr. B (Fy = 35 ksi)

   b. Light Gauge
      i. Light Gauge Truss Steel: ASTM A653 (Fy = 40ksi)
      ii. Light Gauge Bracing & Bridging: ASTM A653 (Fy = 40ksi)
      iii. Light Gauge Coating: Protective Zinc, Class G60 min

   c. Concrete
      i. Exterior slabs on grade: f`c = 4,000 psi, air entrained
      ii. Foundations: f`c = 4,000 psi, air entrained
      iii. Concrete fill & sidewalks: f`c = 3000psi

   d. Unit Masonry:
      i. f`m = 1500 psi min
      ii. Mortar: ASTM 270
      iii. Premix Mortar: ASTM C 387, Type M
      iv. Concrete Grout: 3/8" Pearock min
         1. f`c = 3000pci
      v. Exterior Wall Horizontal Reinforcement: @ 16" Centers vertically min

   e. Reinforcing Steel: ASTM A615-05, Gr. 60

   f. Welded Wire Fabric: ASTM A185

   g. Metal Grating
      i. Roof deck – ASTM A653, Gr.33, 33 ksi, 22 gauge min.

   h. Grout – ASTM C1107

   i. Cast in Place Concrete Anchors
      i. Bolts – ASTM F1554
      ii. Nuts – ASTM F563
      iii. Washers – ASTM F436

   j. Drilled-in Anchors: Based in *Hilti Co.*

ATC Hazards by Location

Search Information

Coordinates:	30.27435833, -82.79940833
Elevation:	125 ft
Timestamp:	2020-04-28T16:46:28.021Z
Hazard Type:	Snow

ASCE 7-16

Ground Snow Load .... 0 lb/sqft

ASCE 7-10

Ground Snow Load .... 0 lb/sqft

5psf per bid documents

ASCE 7-05

Ground Snow Load .... 0 lb/sqft

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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ATC Hazards by Location

Search Information

Coordinates:	30.27435833, -82.79940833
Elevation:	125 ft
Timestamp:	2020-04-28T16:44:58.922Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	73 mph
MRI 25-Year	82 mph
MRI 50-Year	88 mph
MRI 100-Year	96 mph
Risk Category I	107 mph
Risk Category II	117 mph
Risk Category III	127 mph
Risk Category IV	131 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	97 mph
Risk Category I	108 mph
Risk Category II	118 mph
Risk Category III-IV	127 mph

120mph per bid documents

ASCE 7-05

ASCE 7-05 Wind Speed	100 mph

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

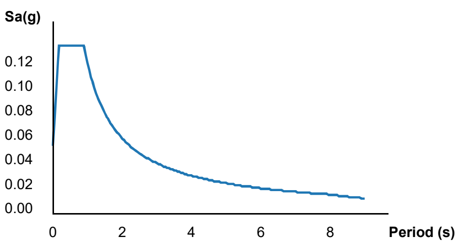
Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

ATC Hazards by Location

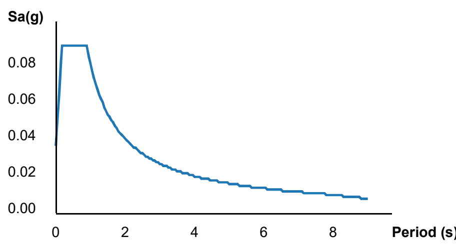
Search Information

Coordinates:	30.27435833, -82.79940833
Elevation:	125 ft
Timestamp:	2020-04-28T16:47:38.473Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.086	MCE_R ground motion (period=0.2s)
S_1	0.051	MCE_R ground motion (period=1.0s)
S_{MS}	0.137	Site-modified spectral acceleration value
S_{M1}	0.123	Site-modified spectral acceleration value
S_{DS}	0.092	Numeric seismic design value at 0.2s SA
S_{D1}	0.082	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.91	Coefficient of risk (0.2s)
CR_1	0.891	Coefficient of risk (1.0s)
PGA	0.041	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.065	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	0.086	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.094	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.051	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.057	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

Wind Pressure Calculations

Purpose: This worksheet calculates the wind force for the center pavillion

References: 1. FBC 2017 , 6th Edition
2. ASCE/SEI 7-10

Wind Load (Restrooms):

L_{bldg} **62**ft B_{bldg} **40**ft H_{eave} **18.094**ft H_{mean} **25.25**ft

Loads are generated according to Chapter 29 - MWFRS Envelope Procedure

Building Risk Category is II Table 1.5-1

Building is Partially Enclosed

V_w **120**mph Basic Wind Speed from Figure 1609.3 (1)

K_d **0.85** Wind Directionality Factor from Table 26.6-1
for Buildings - Components & Cladding

Exposure Category is Exposure "C"

K_{zt} **1.0** Topographic Factor, no topographic adjustment, Section 26.8.2

G_f **0.85** Gust response factor per Section 26.9.1

K_z **0.94** Velocity pressure exposure coefficient from Table 28.3-1

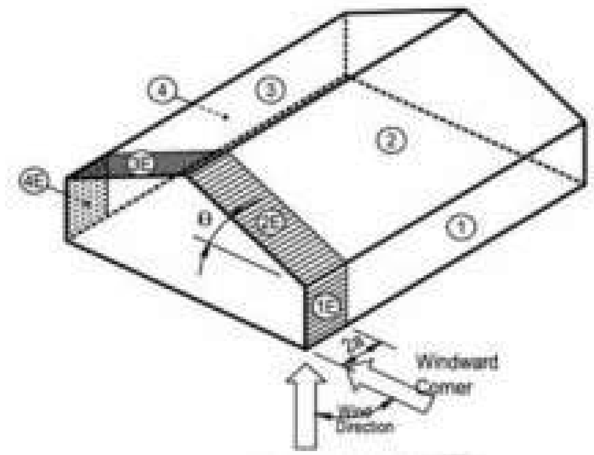
GC_{pip} **0.55** Internal pressure coeffcient (plus or minus) Table 26.11-1

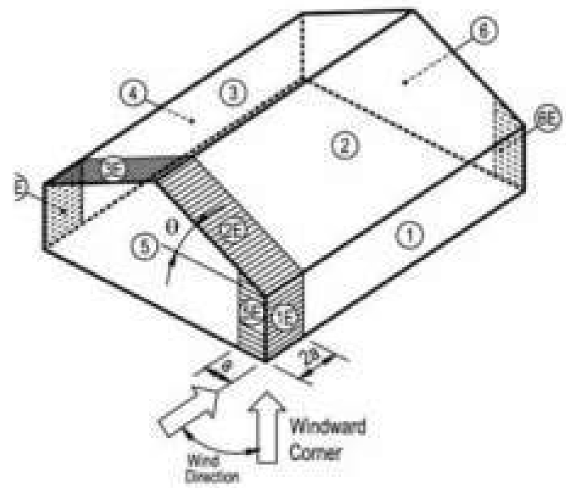
GC_{pin} **0.55**

$$q_z \quad .00256\ K_z\ K_{zt}\ K_d\ \frac{V_w^{\,2}}{1\text{mph}^2}\ 1\text{psf} \quad 29.454\ \text{psf}$$

Velocity base pressure at the level of the shroud
Equation 28.3-1

Refer to Chapter 28, Wind Loads on Buildings: MWFRS - Envelope Procedure





Load Case A

Load Case B

$Roof_{slope}$ $\frac{\mathbf{8}}{\mathbf{12}}$ **0.667** θ atan $Roof_{slope}$ **33.69** deg

a_1 **0.1** B_{bldg} **4** ft a_2 **0.4** H_{mean} **10.1** ft

a min a_1 a_2 **4** ft

Pressures, Figure 28.4-1

Roof Angle θ (degrees)	LOAD CASE A							
	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Table 1 GC_P

GC_{pA1} **0.56** GC_{pA1E} **0.69**

GC_{pA2} **0.21** GC_{pA2E} **0.27**

GC_{pA3} **0.43** GC_{pA3E} **0.53**

GC_{pA4} **0.37** GC_{pA4E} **0.48**

p_{A1} q_z GC_{pA1} GC_{pin} **32.694** psf — Windward Wall Pressure

$p_{A1.1}$ q_z GC_{pA1} GC_{pip} **0.295** psf — Windward Wall Pressure

p_{A2} q_z GC_{pA2} GC_{pin} **22.385** psf — Windward Roof Pressure

$p_{A2.1}$ q_z GC_{pA2} GC_{pip} **10.014** psf — Windward Roof Pressure

p_{A3} q_z GC_{pA3} GC_{pin} **3.535** psf — Leeward Roof Pressure

$p_{A3.1}$ q_z GC_{pA3} GC_{pip} **28.865** psf — Leeward Roof Pressure

P_{A4} q_z GC_{pA4} GC_{pin} **5.302** psf — Leeward Wall Pressure

$P_{A4.1}$ q_z GC_{pA4} GC_{pip} **27.098** psf — Leeward Wall Pressure

Roof Angle θ (degrees)	LOAD CASE B											
	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43

Table 2 GC_P

GC_{pB1}	**0.45**	GC_{pB1E}	**0.48**
GC_{pB2}	**0.69**	GC_{pB2E}	**1.07**
GC_{pB3}	**0.37**	GC_{pB3E}	**0.53**
GC_{pB4}	**0.45**	GC_{pB4E}	**0.48**
GC_{pB5}	**0.40**	GC_{pB5E}	**0.61**
GC_{pB6}	**0.29**	GC_{pB6E}	**0.43**

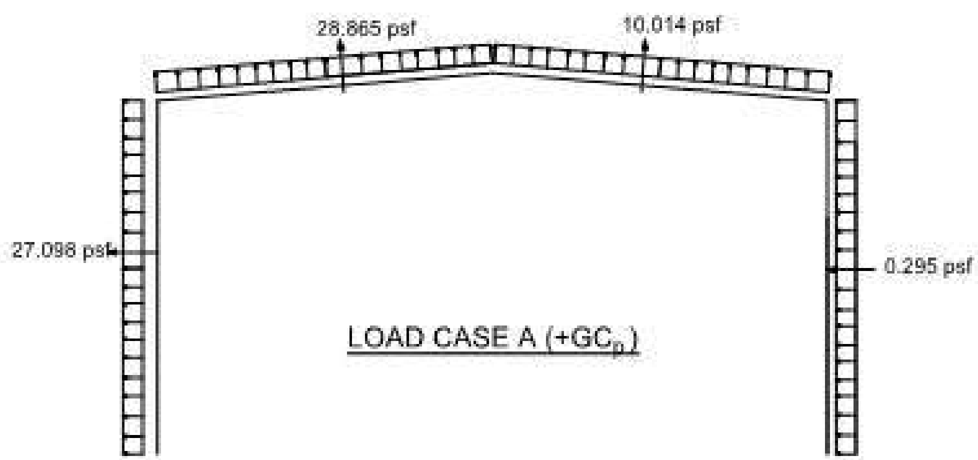
p_{B1} q_z GC_{pB1} GC_{pin} **2.945** psf — Side Wall Pressure

$p_{B1.1}$ q_z GC_{pB1} GC_{pip} **29.454** psf — Side Wall Pressure

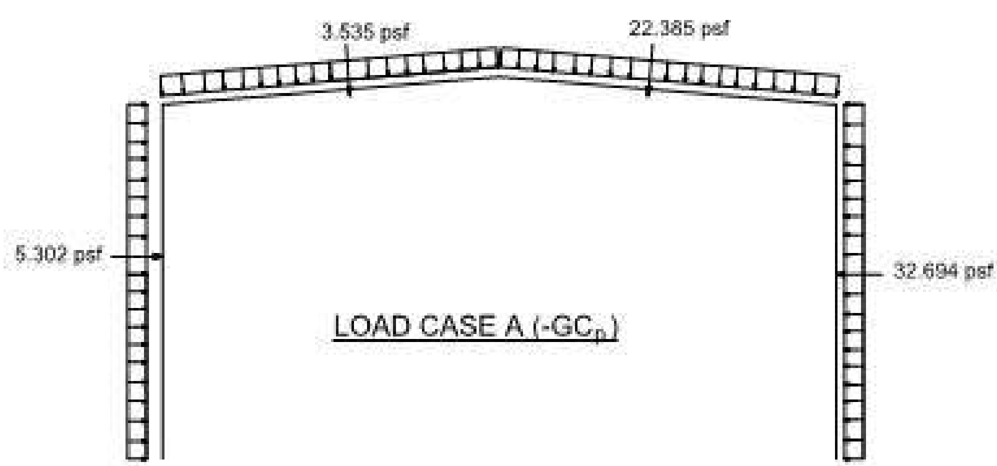
p_{B2} q_z GC_{pB2} GC_{pin} **4.124** psf — Side Roof Pressure

$p_{B2.1}$ q_z GC_{pB2} GC_{pip} **36.523** psf — Side Roof Pressure

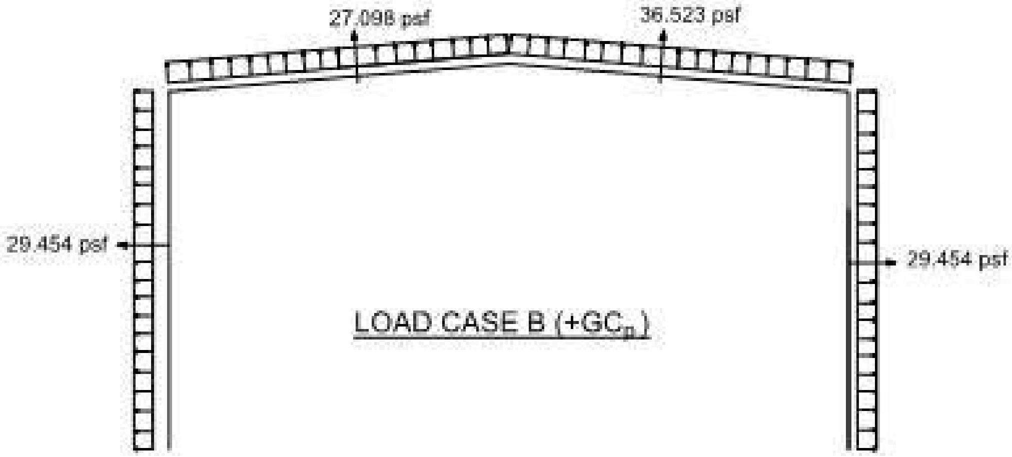
p_{B3}	q_z	GC_{pB3}	GC_{pin}	**5.302** psf	Side Roof Pressure
$p_{B3.1}$	q_z	GC_{pB3}	GC_{pip}	**27.098** psf	Side Roof Pressure
P_{B4}	q_z	GC_{pB4}	GC_{pin}	**2.945** psf	Side Wall Pressure
$P_{B4.1}$	q_z	GC_{pB4}	GC_{pip}	**29.454** psf	Side Wall Pressure
p_{B5}	q_z	GC_{pB5}	GC_{pin}	**27.982** psf	Windward Wall Pressure
$p_{B5.1}$	q_z	GC_{pB5}	GC_{pip}	**4.418** psf	Windward Wall Pressure
P_{B6}	q_z	GC_{pB6}	GC_{pin}	**7.658** psf	Leeward Wall Pressure
$P_{B6.1}$	q_z	GC_{pB6}	GC_{pip}	**24.742** psf	Leeward Wall Pressure

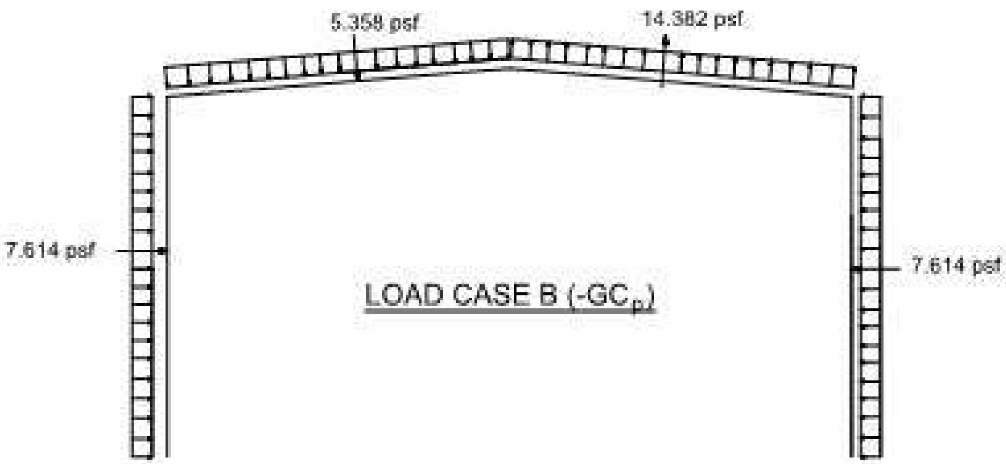


LOAD CASE A (+GC_p)



LOAD CASE A (-GC_p)


27.098 psf
36.523 psf
29.454 psf
29.454 psf
LOAD CASE B (+GCp)


5.358 psf
14.382 psf
7.614 psf
7.614 psf
LOAD CASE B (-GCp)

Purpose: This worksheet calculates the wind force for the restrooms

References: 1. FBC 2017 , 6th Edition
2. ASCE/SEI 7-10

Wind Load (Restrooms):

L_{bldg} **89**ft **3**in B_{bldg} **30**ft H_{eave} **14.22**ft H_{mean} **19.11**ft

Loads are generated according to Chapter 29 - MWFRS Envelope Procedure

Building Risk Category is II Table 1.5-1

Building is Enclosed

V_w **120**mph — Basic Wind Speed from Ref. 1, Figure 1609.5.3 (1)

K_d **0.85** — Wind Directionality Factor from Table 26.6-1 for Buildings - Components & Cladding

Exposure Category is Exposure "C"

K_{zt} **1.0** — Topographic Factor, no topographic adjustment, Section 26.8.2

G_f **0.85** — Gust response factor per Section 26.9.1

K_z **0.9** — Velocity pressure exposure coefficient from Table 28.3-1

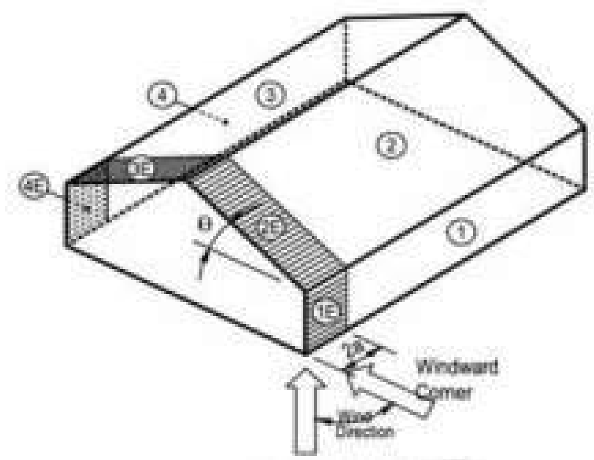
GC_{pip} **0.18** — Internal pressure coeffcient (plus or minus) Table 26.11-1

GC_{pin} **0.18**

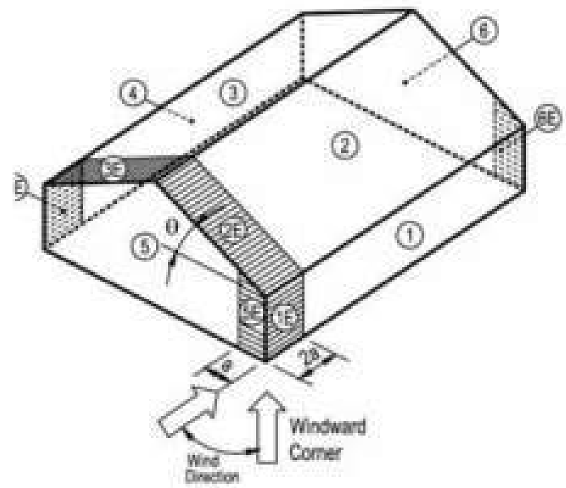
$$q_z \quad .00256\, K_z\, K_{zt}\, K_d \frac{V_w^{\,2}}{1\,\text{mph}^2}\, 1\,\text{psf} \quad 28.201\ \text{psf}$$

Velocity base pressure at the level of the shroud Equation 28.3-1

Refer to Chapter 28, Wind Loads on Buildings: MWFRS - Envelope Procedure



Load Case A



Load Case B

Roof_{slope} $\mathbf{\frac{8}{12}}$ $\mathbf{0.667}$ $\qquad$ θ $\text{atan}\ \text{Roof}_{slope}$ $\mathbf{33.69}$ deg

a_1 $\mathbf{0.1}\ B_{bldg}$ $\mathbf{3}$ ft $\qquad$ a_2 $\mathbf{0.4}\ H_{mean}$ $\mathbf{7.644}$ ft

a $\min\ a_1\ a_2$ $\mathbf{3}$ ft

Pressures, Figure 28.4-1

Roof Angle θ (degrees)	LOAD CASE A							
	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Table 1 GC_P

GC_{pA1} $\mathbf{0.56}$ $\qquad$ GC_{pA1E} $\mathbf{0.69}$

GC_{pA2} $\mathbf{0.21}$ $\qquad$ GC_{pA2E} $\mathbf{0.27}$

GC_{pA3} $\mathbf{0.43}$ $\qquad$ GC_{pA3E} $\mathbf{0.53}$

GC_{pA4} $\mathbf{0.37}$ $\qquad$ GC_{pA4E} $\mathbf{0.48}$

p_{A1} q_z GC_{pA1} GC_{pin} **20.869** psf — Windward Wall Pressure

$p_{A1.1}$ q_z GC_{pA1} GC_{pip} **10.716** psf — Windward Wall Pressure

p_{A2} q_z GC_{pA2} GC_{pin} **10.998** psf — Windward Roof Pressure

$p_{A2.1}$ q_z GC_{pA2} GC_{pip} **0.846** psf — Windward Roof Pressure

p_{A3} q_z GC_{pA3} GC_{pin} **7.05** psf — Leeward Roof Pressure

$p_{A3.1}$ q_z GC_{pA3} GC_{pip} **17.203** psf — Leeward Roof Pressure

P_{A4} q_z GC_{pA4} GC_{pin} **5.358** psf — Leeward Wall Pressure

$P_{A4.1}$ q_z GC_{pA4} GC_{pip} **15.511** psf — Leeward Wall Pressure

Roof Angle θ (degrees)	LOAD CASE B											
	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43

Table 2 GC_P

GC_{pB1}	**0.45**	GC_{pB1E}	**0.48**
GC_{pB2}	**0.69**	GC_{pB2E}	**1.07**
GC_{pB3}	**0.37**	GC_{pB3E}	**0.53**
GC_{pB4}	**0.45**	GC_{pB4E}	**0.48**
GC_{pB5}	**0.40**	GC_{pB5E}	**0.61**
GC_{pB6}	**0.29**	GC_{pB6E}	**0.43**

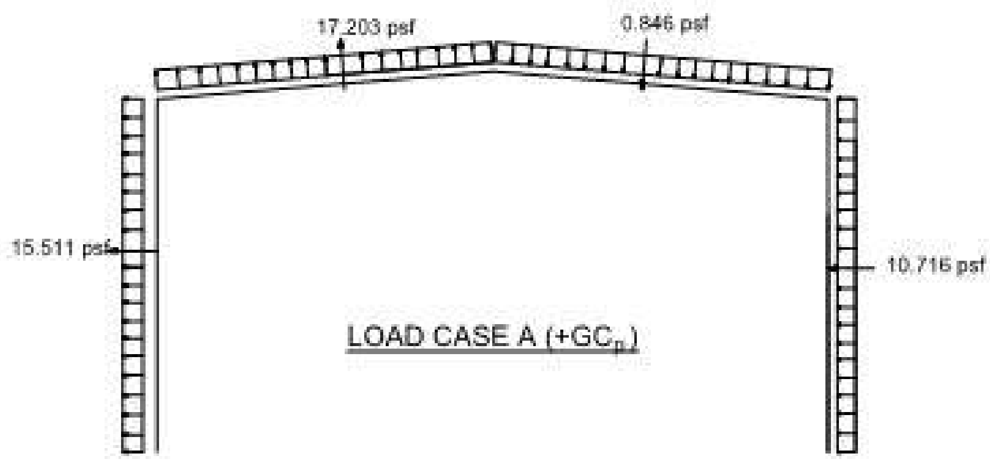
p_{B1} q_z GC_{pB1} GC_{pin} **7.614** psf — Side Wall Pressure

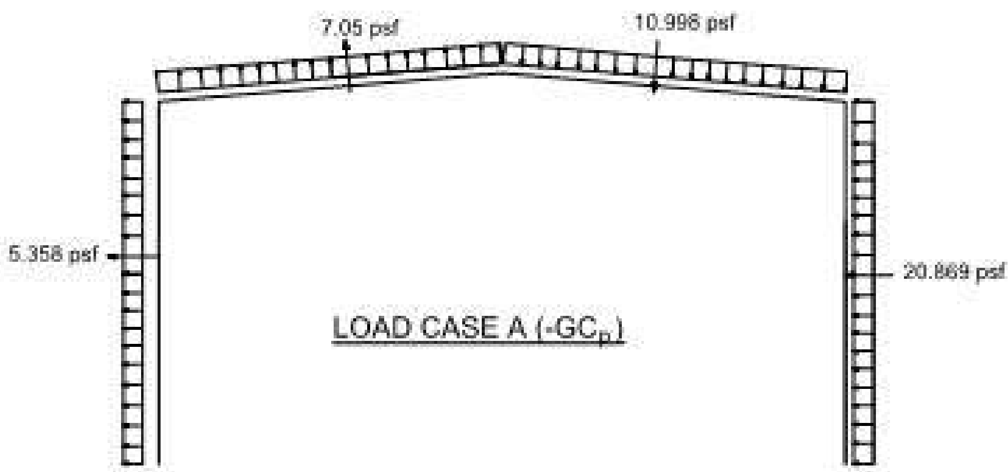
$p_{B1.1}$ q_z GC_{pB1} GC_{pip} **17.767** psf — Side Wall Pressure

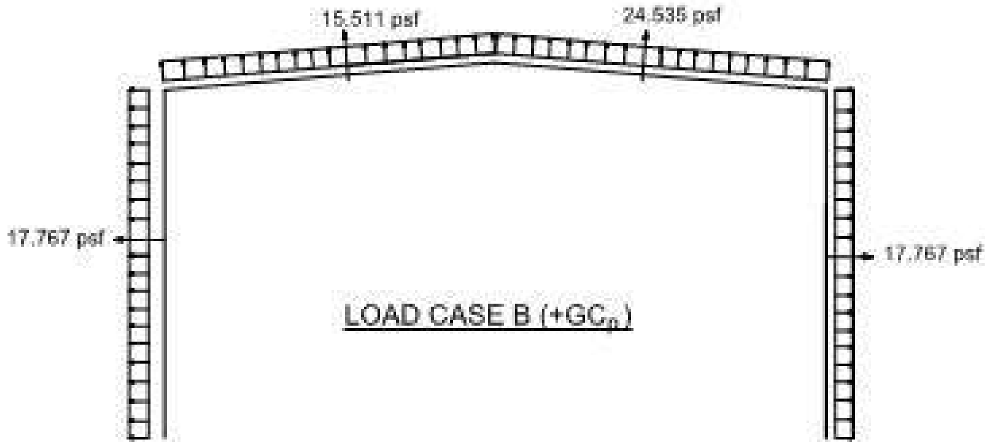
p_{B2} q_z GC_{pB2} GC_{pin} **14.382** psf — Side Roof Pressure

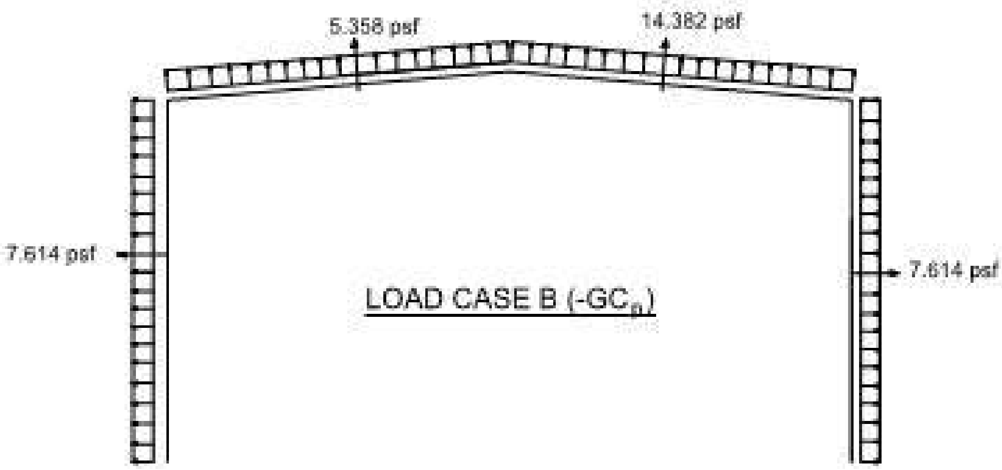
$p_{B2.1}$ q_z GC_{pB2} GC_{pip} **24.535** psf — Side Roof Pressure

p_{B3}	q_z	GC_{pB3}	GC_{pin}	**5.358** psf	Side Roof Pressure
$p_{B3.1}$	q_z	GC_{pB3}	GC_{pip}	**15.511** psf	Side Roof Pressure
P_{B4}	q_z	GC_{pB4}	GC_{pin}	**7.614** psf	Side Wall Pressure
$P_{B4.1}$	q_z	GC_{pB4}	GC_{pip}	**17.767** psf	Side Wall Pressure
p_{B5}	q_z	GC_{pB5}	GC_{pin}	**16.357** psf	Windward all Pressure
$p_{B5.1}$	q_z	GC_{pB5}	GC_{pip}	**6.204** psf	Windward Wall Pressure
P_{B6}	q_z	GC_{pB6}	GC_{pin}	**3.102** psf	Leeward Wall Pressure
$P_{B6.1}$	q_z	GC_{pB6}	GC_{pip}	**13.254** psf	Leeward Wall Pressure






15.511 psf
24.535 psf
17.767 psf
17.767 psf
LOAD CASE B (+GCpi)


5.358 psf
14.382 psf
7.614 psf
7.614 psf
LOAD CASE B (-GCpi)

Purpose: This worksheet calculates the wind force for the storage building

References: 1. FBC 2017 , 6th Edition
2. ASCE/SEI 7-10

Wind Load :

L_{bldg} **18**ft B_{bldg} **20**ft H_{eave} **10**ft **8**in H_{mean} **12**ft **8**in

Loads are generated according to Chapter 28 - MWFRS Envelope Procedure

Building Risk Category is II Table 1.5-1

Building is Enclosed

V_w **120**mph — Basic Wind Speed from Ref. 1, Figure 1609.3 (1)

K_d **0.85** — Wind Directionality Factor from Table 26.6-1 for Buildings - Components & Cladding

Exposure Category is Exposure "C"

K_{zt} **1.0** — Topographic Factor, no topographic adjustment, Section 26.8.2

G_f **0.85** — Gust response factor per Section 26.9.1

K_z **0.85** — Velocity pressure exposure coefficient from Table 28.3-1

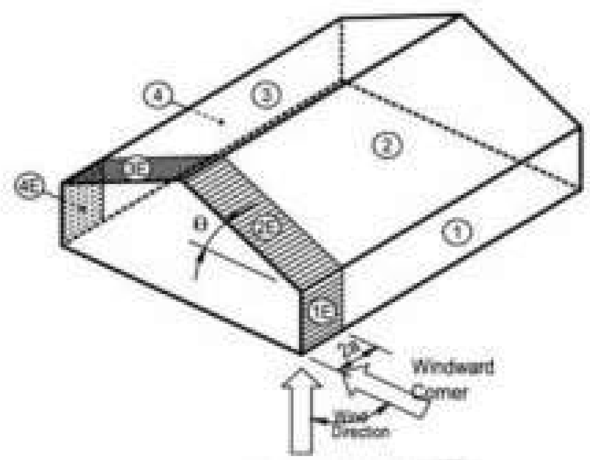
GC_{pip} **0.18** — Internal pressure coeffcient (plus or minus) Table 26.11-1

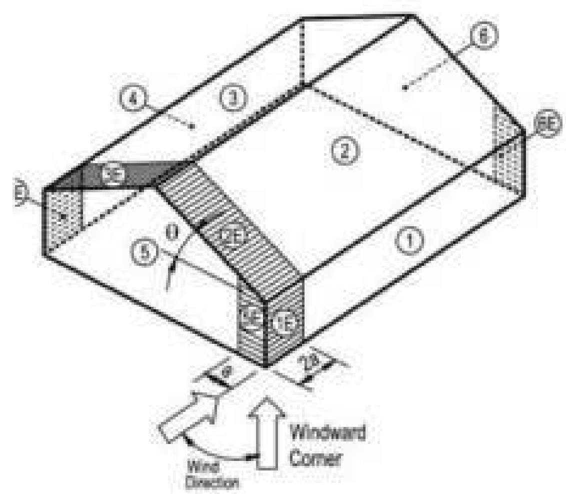
GC_{pin} **0.18**

$$q_z \quad .00256\, K_z\, K_{zt}\, K_d \frac{V_w^{\,2}}{1\text{mph}^2}\, 1\text{psf} \quad 26.634\ \text{psf}$$

Velocity base pressure at the level of the shroud
Equation 28.3-1

Refer to Chapter 28, Wind Loads on Buildings: MWFRS - Envelope Procedure





Load Case A

Load Case B

Roof_{slope} $\frac{\mathbf{8}}{\mathbf{12}}$ **0.667** θ $\text{atan}\ \text{Roof}_{slope}$ **33.69** deg

a_1 **0.1** B_{bldg} **2** ft a_2 **0.4** H_{mean} **5.067** ft

a $\min\ a_1\ a_2$ **2** ft

Pressures, Figure 28.4-1

Roof Angle θ (degrees)	LOAD CASE A							
	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Table 1 GC_P

GC_{pA1} **0.56** GC_{pA1E} **0.69**

GC_{pA2} **0.21** GC_{pA2E} **0.27**

GC_{pA3} **0.43** GC_{pA3E} **0.53**

GC_{pA4} **0.37** GC_{pA4E} **0.48**

p_{A1} q_z GC_{pA1} GC_{pin} **19.709** psf — Windward Wall Pressure

$p_{A1.1}$ q_z GC_{pA1} GC_{pip} **10.121** psf — Windward Wall Pressure

p_{A2} q_z GC_{pA2} GC_{pin} **10.387** psf — Windward Roof Pressure

$p_{A2.1}$ q_z GC_{pA2} GC_{pip} **0.799** psf — Windward Roof Pressure

p_{A3} q_z GC_{pA3} GC_{pin} **6.659** psf — Leeward Roof Pressure

$p_{A3.1}$ q_z GC_{pA3} GC_{pip} **16.247** psf — Leeward Roof Pressure

P_{A4} q_z GC_{pA4} GC_{pin} **5.061** psf — Leeward Wall Pressure

$P_{A4.1}$ q_z GC_{pA4} GC_{pip} **14.649** psf — Leeward Wall Pressure

Roof Angle θ (degrees)	LOAD CASE B											
	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43

Table 2 GC_P

GC_{pB1}	**0.45**	GC_{pB1E}	**0.48**
GC_{pB2}	**0.69**	GC_{pB2E}	**1.07**
GC_{pB3}	**0.37**	GC_{pB3E}	**0.53**
GC_{pB4}	**0.45**	GC_{pB4E}	**0.48**
GC_{pB5}	**0.40**	GC_{pB5E}	**0.61**
GC_{pB6}	**0.29**	GC_{pB6E}	**0.43**

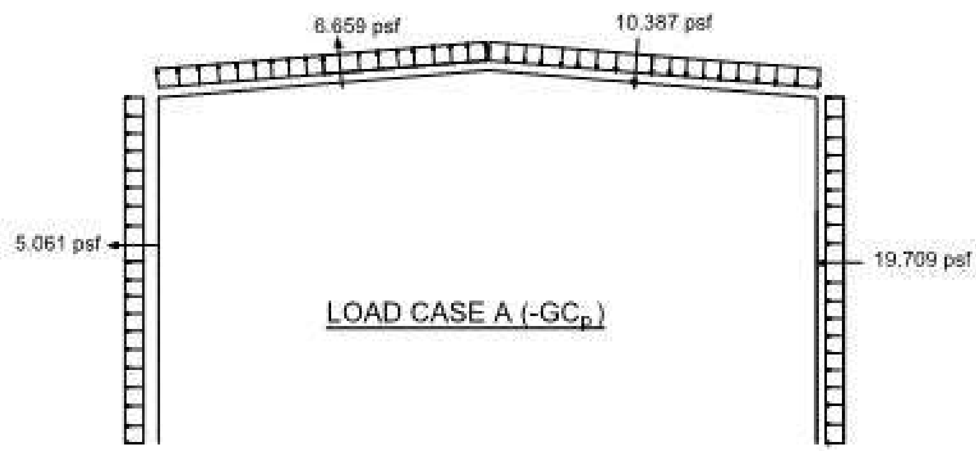
p_{B1} q_z GC_{pB1} GC_{pin} **7.191** psf — Side Wall Pressure

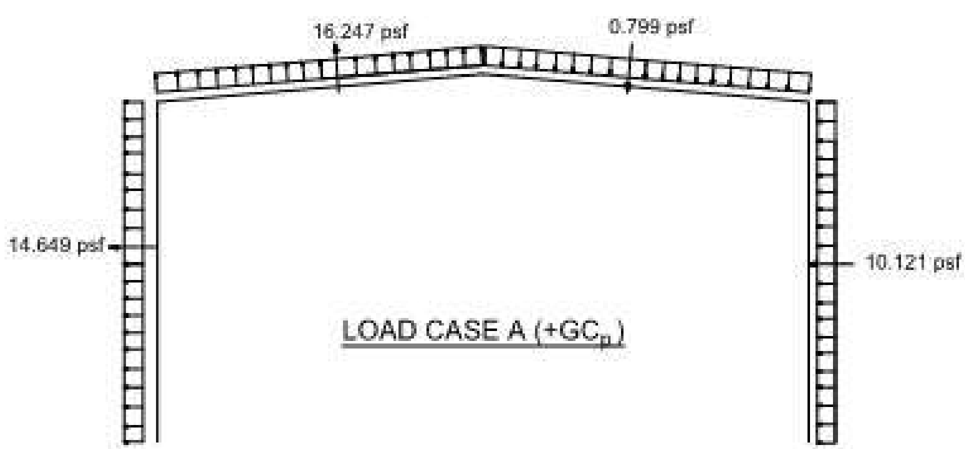
$p_{B1.1}$ q_z GC_{pB1} GC_{pip} **16.78** psf — Side Wall Pressure

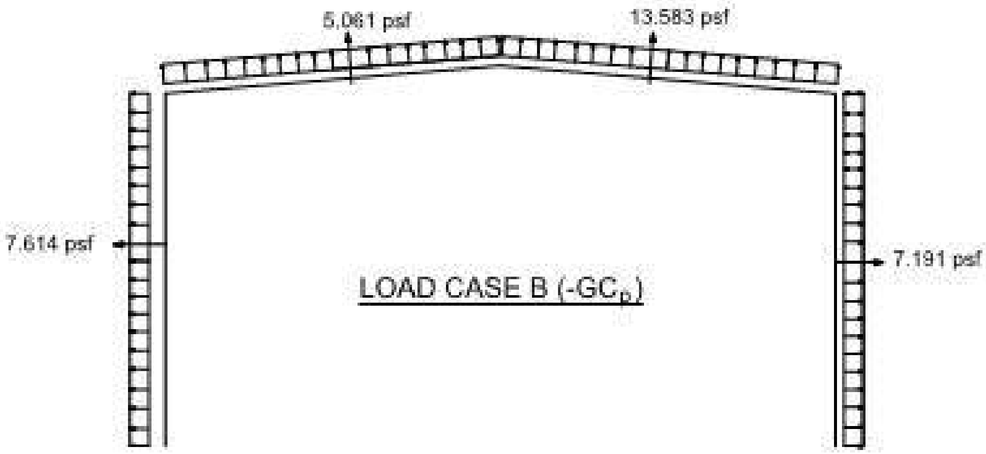
p_{B2} q_z GC_{pB2} GC_{pin} **13.583** psf — Side Roof Pressure

$p_{B2.1}$ q_z GC_{pB2} GC_{pip} **23.172** psf — Side Roof Pressure

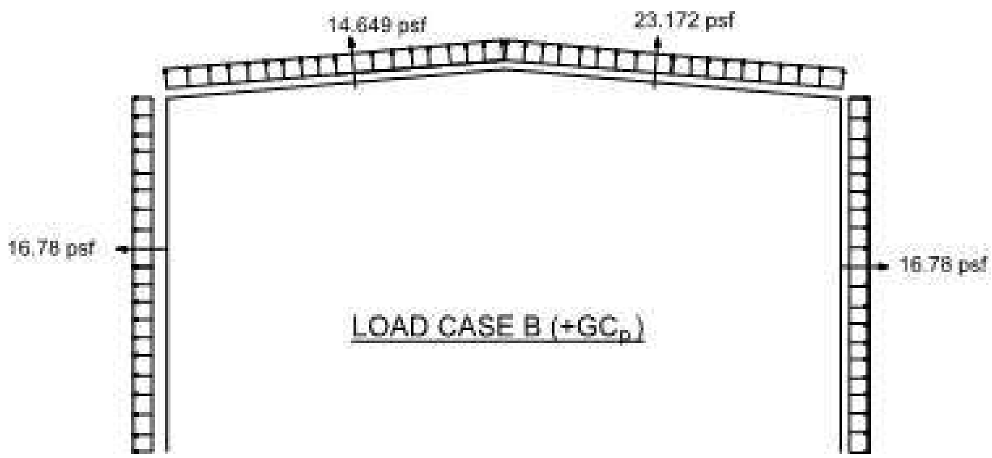
p_{B3}	q_z	GC_{pB3}	GC_{pin}	**5.061** psf	Side Roof Pressure
$p_{B3.1}$	q_z	GC_{pB3}	GC_{pip}	**14.649** psf	Side Roof Pressure
P_{B4}	q_z	GC_{pB4}	GC_{pin}	**7.191** psf	Side Wall Pressure
$P_{B4.1}$	q_z	GC_{pB4}	GC_{pip}	**16.78** psf	Side Wall Pressure
p_{B5}	q_z	GC_{pB5}	GC_{pin}	**15.448** psf	Windward Wall Pressure
$p_{B5.1}$	q_z	GC_{pB5}	GC_{pip}	**5.86** psf	Windward Wall Pressure
P_{B6}	q_z	GC_{pB6}	GC_{pin}	**2.93** psf	Leeward Wall Pressure
$P_{B6.1}$	q_z	GC_{pB6}	GC_{pip}	**12.518** psf	Leewar Wal Pressure







LOAD CASE B (-GC_p)



LOAD CASE B (+GC_p)

Purpose: This worksheet calculates the wind force for the picnic pavilion

References: 1. FBC 2017 , 6th Edition
2. ASCE/SEI 7-10

Wind Load (Restrooms):

L_{bldg} **26**ft B_{bldg} **14**ft H_{eave} **9**ft **8**in H_{mean} **12**ft

Loads are generated according to Chapter 29 - MWFRS Envelope Procedure

Building Risk Category is II Table 1.5-1

Structure is Open

V_w **120**mph Basic Wind Speed from Ref. 1, Figure 1609.3 (1)

K_d **0.85** Wind Directionality Factor from Table 26.6-1 for Buildings - MWFRS

Exposure Category is Exposure "C"

K_{zt} **1.0** Topographic Factor, no topographic adjustment, Section 26.8.2

G_f **0.85** Gust response factor per Section 26.9.1

K_z **0.85** Velocity pressure exposure coefficient from Table 28.3-1

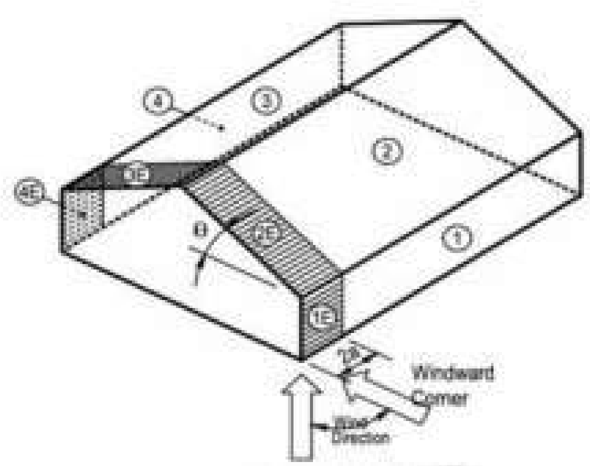
GC_{pip} **0** Internal pressure coeffcient (plus or minus) Table 26.11-1

GC_{pin} **0**

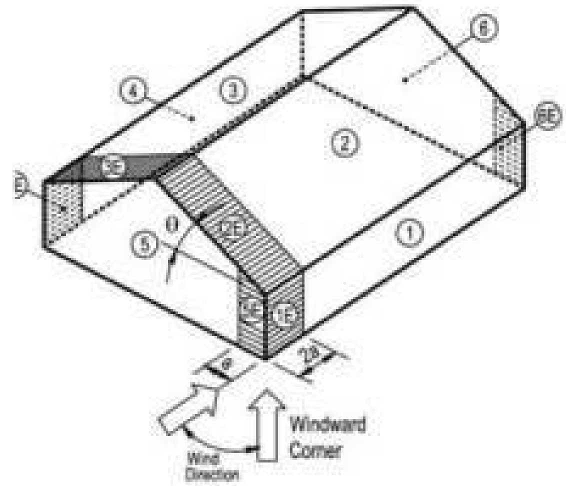
$$q_z \quad .00256\, K_z\, K_{zt}\, K_d\, \frac{V_w^2}{1\,\mathrm{mph}^2}\, 1\,\mathrm{psf} \quad 26.634\ \mathrm{psf}$$

Velocity base pressure at the level of the shroud Equation 28.3-1

Refer to Chapter 27, Wind Loads on Buildings: MWFRS - Directional Procedure



Load Case A

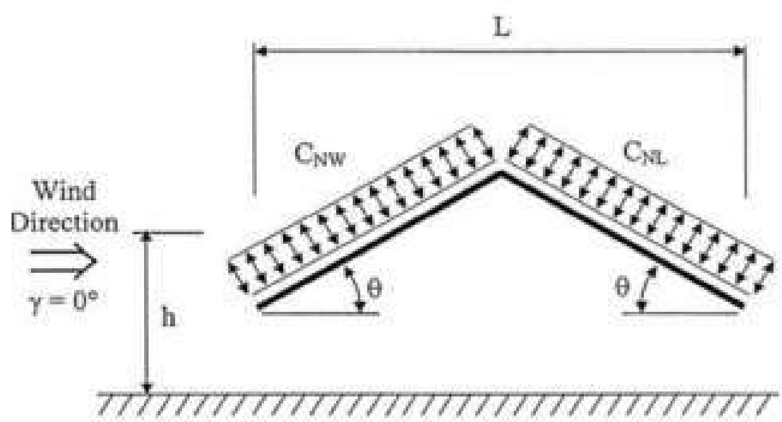


Load Case B

$\text{Roof}_{slope} \quad \dfrac{\mathbf{8}}{\mathbf{12}} \quad \mathbf{0.667}$ $\qquad \theta \quad \text{atan}\ \text{Roof}_{slope} \quad \mathbf{33.69}\ \text{deg}$

$a_1 \quad \mathbf{0.1}\ B_{bldg} \quad \mathbf{1.4}\ \text{ft}$ $\qquad a_2 \quad \mathbf{0.4}\ H_{mean} \quad \mathbf{4.8}\ \text{ft}$

$a \quad \min\ a_1\ a_2 \quad \mathbf{1.4}\ \text{ft}$



For Pressures, Figure 27.4-5

Roof Angle, θ	Load Case	Wind Direction, $\gamma = 0°, 180°$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	1.1	-0.3	-1.6	-1
	B	0.2	-1.2	-0.9	-1.7
15°	A	1.1	-0.4	-1.2	-1
	B	0.1	-1.1	-0.6	-1.6
22.5°	A	1.1	0.1	-1.2	-1.2
	B	-0.1	-0.8	-0.8	-1.7
30°	A	1.3	0.3	-0.7	-0.7
	B	-0.1	-0.9	-0.2	-1.1
37.5°	A	1.3	0.6	-0.6	-0.6
	B	-0.2	-0.6	-0.3	-0.9
45°	A	1.1	0.9	-0.5	-0.5
	B	-0.3	-0.5	-0.3	-0.7

Notes

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting toward and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.

From note 2, the clear wind values will be used

By interpolation

For load case A: C_{NWA} **1.3** C_{NLA} **0.42**

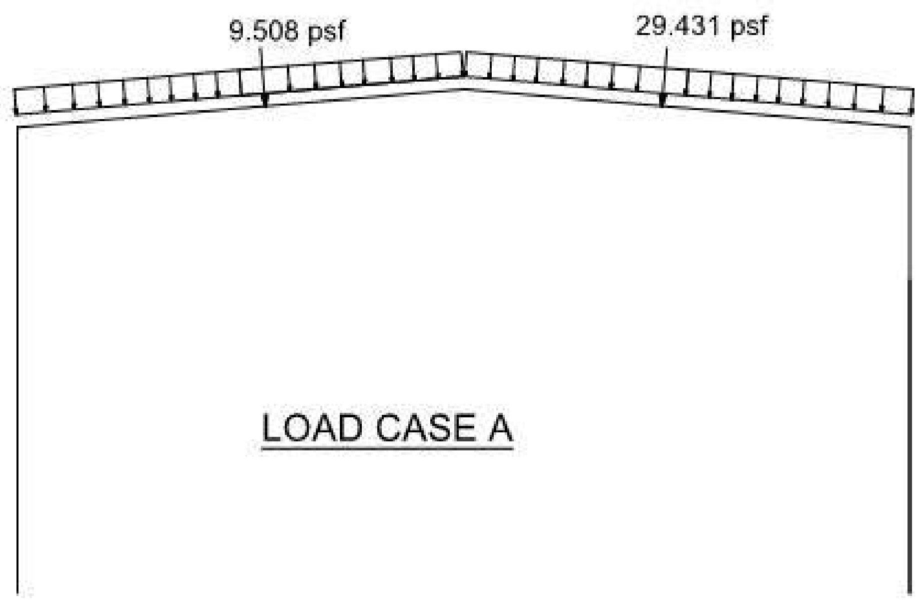
For load case B: C_{NWB} **0.14** C_{NLB} **0.78**

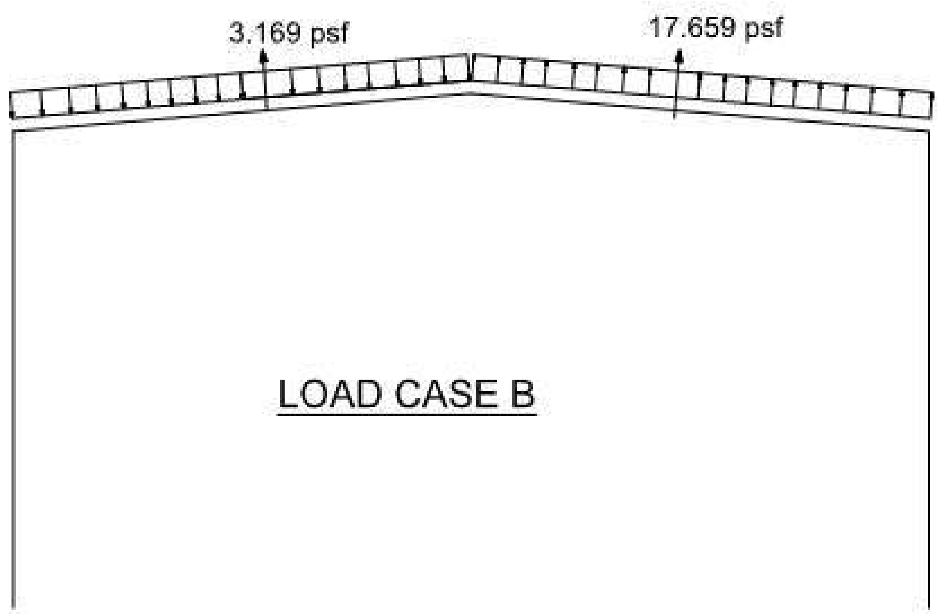
p_{AW} $q_z\ G_f\ C_{NWA}$ **29.431** psf Windward Roof Pressure

p_{AL} $q_z\ G_f\ C_{NLA}$ **9.508** psf Leeward Roof Pressure

p_{BW} $q_z\ G_f\ C_{NWB}$ **3.169** psf Windward Roof Pressure

p_{BL} $q_z\ G_f\ C_{NLB}$ **17.659** psf Leeward Roof Pressure


9.508 psf
29.431 psf
LOAD CASE A


3.169 psf
17.659 psf
LOAD CASE B

SUBJECT: I-10 Rest Areas
Dumpter Area Wind Load
BY: NJA DATE: 5/14/2020
CHCK BY: EJM DATE: 5/26/2020

PROJ. # B190988.00
SHEET NO. 1 of 2

gai consultants

Purpose: This worksheet calculates the wind force for the Dumpster Area Wall

References: 1. FBC 2017 , 6th Edition
2. ASCE/SEI 7-10

Wind Load :

$L := 17\,ft + 4\,in = 17.3\ ft$ $B := 10\,ft + 7.625\,in = 10.6\ ft$ $H := 8\,ft$

Loads are generated according to Chapter 29.4 - Solid Freestanding Walls

Building Risk Category is II Table 1.5-1

$V_w := 120\,mph$ Basic Wind Speed from Figure FBC 2017 Fig. 1609.3(1)

$K_d := 0.85$ Wind Directionality Factor from Table 26.6-1

Exposure Category is Exposure "C"

$K_{zt} := 1.0$ Topographic Factor, no topographic adjustment, Section 26.8.2

$K_z := 0.85$ Velocity pressure exposure coefficient from Table 29.3-1

$q_z := .00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_w}{mph}\right)^2 psf = 26.6\,psf$ Velocity Pressure Equation 29.3-1

$G := 0.85$ Gust response factor per Section 26.9.1

$s := H = 8\ ft$

$A_{s.1} := B \cdot H = 85.1\ ft^2$

$\frac{s}{H} = 1$ $\frac{B}{s} = 1.3$

	C_f, CASE A & CASE B						
Clearance Ratio, s/h	Aspect Ratio, B/s						
	≤ 0.05	0.1	0.2	0.5	1	2	4
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45

$C_f := \frac{1.40 - 1.45}{2 - 1} \cdot \left(\frac{B}{s} - 1\right) + 1.45 = 1.43$ Figure 29.4-1

$F := q_z \cdot G \cdot C_f \cdot A_{s.1} = 2761\ lbf$ Design Wind Load Equation 29.4-1

$A_{s.2} := L \cdot H = \mathbf{138.7}\ ft^2$

$\frac{s}{H} = \mathbf{1}$ $\frac{L}{s} = \mathbf{2.2}$

C_f, CASE A & CASE B							
Clearance Ratio, s/h	Aspect Ratio, B/s						
	≤ 0.05	0.1	0.2	0.5	1	2	4
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45

$C_f := \frac{\mathbf{1.35} - \mathbf{1.40}}{\mathbf{4} - \mathbf{2}} \cdot \left(\frac{L}{s} - \mathbf{2}\right) + \mathbf{1.40} = \mathbf{1.40}$ Figure 29.4-1

$F := q_z \cdot G \cdot C_f \cdot A_{s.2} = \mathbf{4382}\ lbf$ Design Wind Load Equation 29.4-1

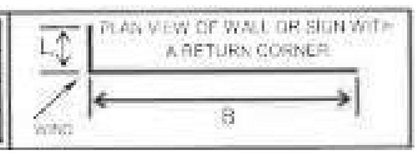
$C_{f.1} := \frac{\mathbf{2.60} - \mathbf{2.25}}{\mathbf{3} - \mathbf{2}} \cdot \left(\frac{L}{s} - \mathbf{2}\right) + \mathbf{2.25} = \mathbf{2.31}$ Figure 29.4-1

$C_{f.2} := \frac{\mathbf{1.7} - \mathbf{1.5}}{\mathbf{3} - \mathbf{2}} \cdot \left(\frac{L}{s} - \mathbf{2}\right) + \mathbf{1.5} = \mathbf{1.53}$ Figure 29.4-1

$C_{f.3} := \mathbf{1.15}$ Figure 29.4-1

*Values shall be multiplied by the following reduction factor when a return corner is present:

L_r/s	Reduction Factor
0.3	0.90
1.0	0.75
≥2	0.60



$\frac{B}{s} = \mathbf{1.3}$

$C_{f.1} := C_{f.1} \cdot \left(\mathbf{1.8} - \frac{s}{H}\right) \cdot \mathbf{.75} = \mathbf{1.39}$ Figure 29.4-1 Note 4 and Return corner note

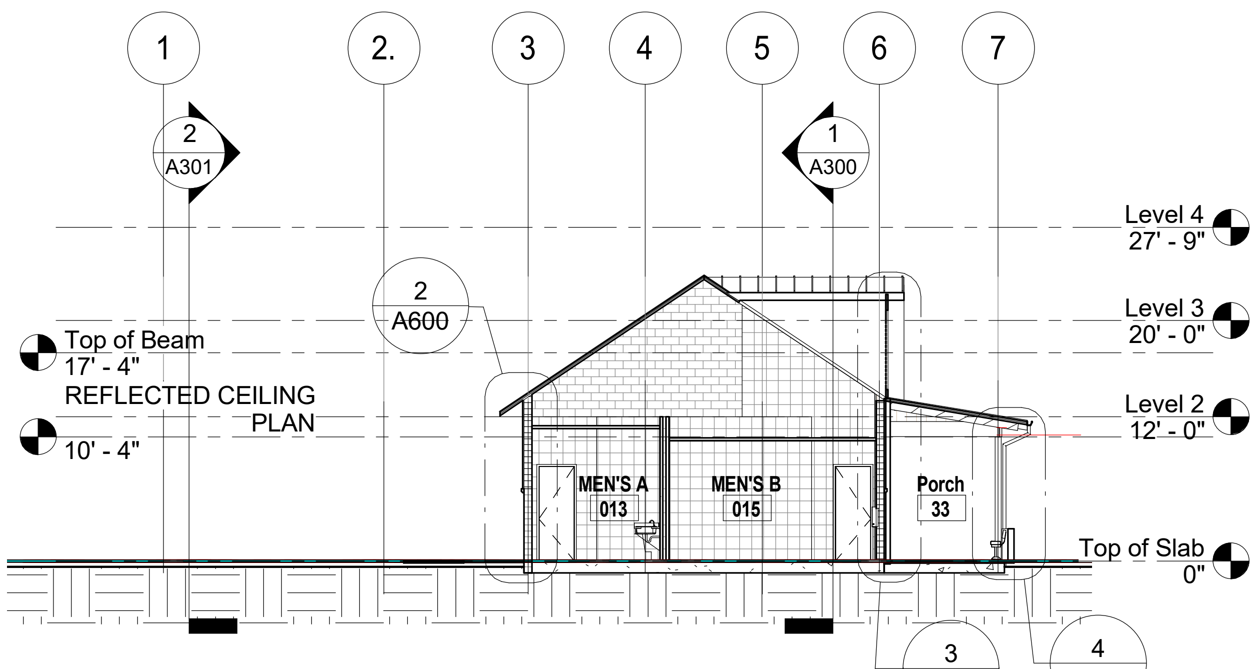
$C_{f.2} := C_{f.2} \cdot \left(\mathbf{1.8} - \frac{s}{H}\right) \cdot \mathbf{.75} = \mathbf{0.92}$ Figure 29.4-1 Note 4 and Return corner note

$C_{f.3} := C_{f.3} \cdot \left(\mathbf{1.8} - \frac{s}{H}\right) \cdot \mathbf{.75} = \mathbf{0.69}$ Figure 29.4-1 Note 4 and Return corner note

$F := q_z \cdot G \cdot C_{f.1} \cdot (H \cdot s) + q_z \cdot G \cdot C_{f.2} \cdot (H \cdot s) + q_z \cdot G \cdot C_{f.2} \cdot \left(A_{s.2} - 2H \cdot s\right) = \mathbf{3562}\ lbf$ Design Wind Load Equation 29.4-1

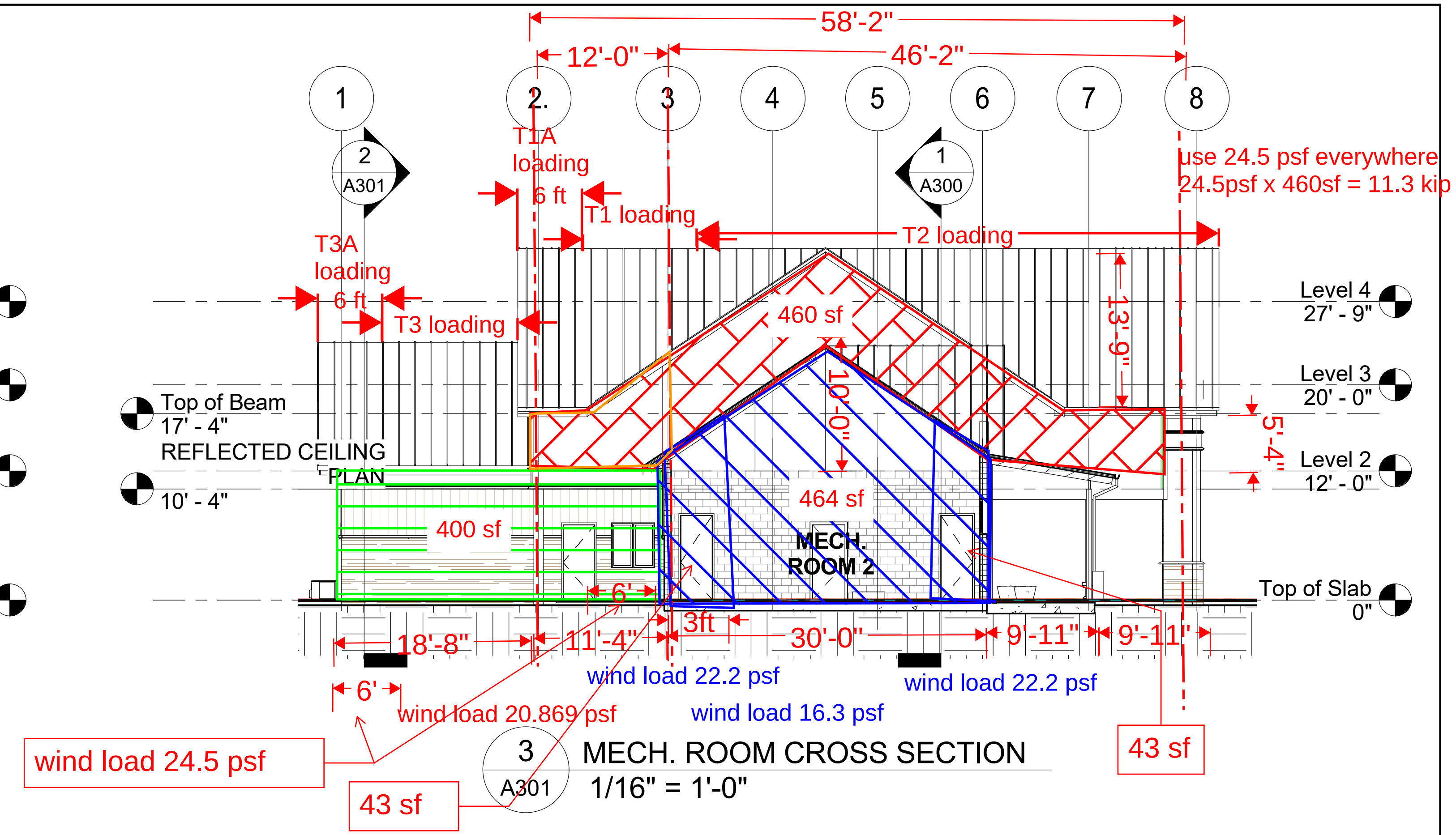
ATTACHMENT B
Rest Area Analysis and Design

Rest Room Building Loading

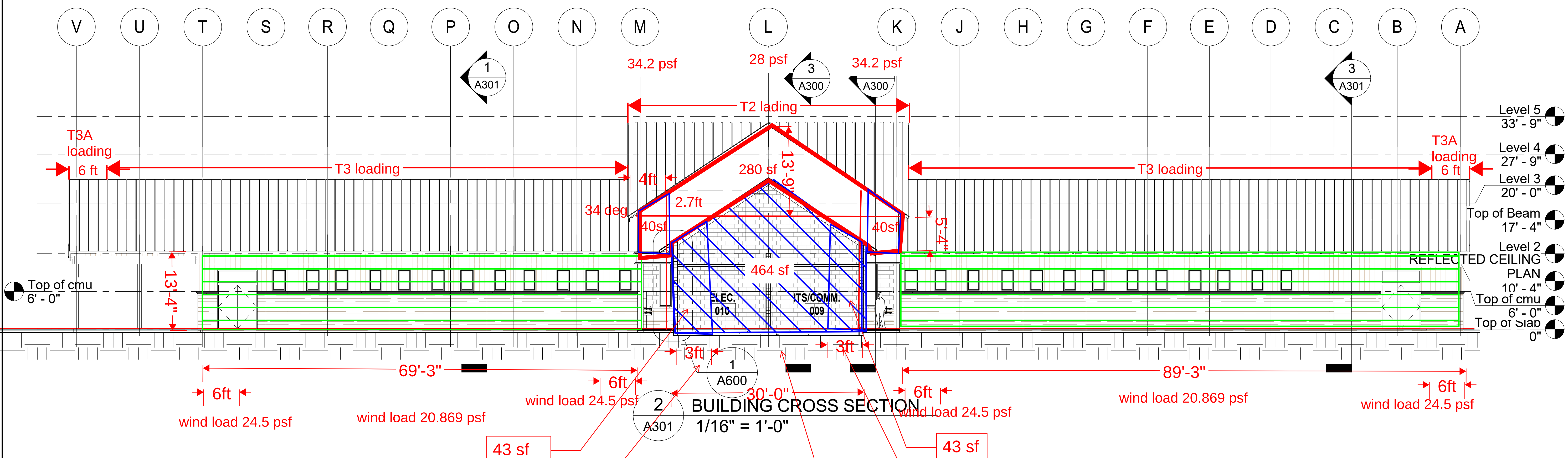


1 A301 RESTROOM CROSS SECTION
1/16" = 1'-0"

For Txx loading see Roof Truss Section



3 A301 MECH. ROOM CROSS SECTION
1/16" = 1'-0"



2 A301 BUILDING CROSS SECTION
1/16" = 1'-0"

C:\Users\simm23411\OneDrive - Atkins Ltd\Documents\10\Final Rev. Model 062018\082918\Final - I-10 082118 R16.rvt

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.
CERTIFICATE OF AUTHORIZATION NO. 24
Jeffrey N. Warmington, RA. NO. 13828

STATE OF FLORIDA
DEPARTMENT

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE AND COLUMBIA	438608-1 and 438609-1

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

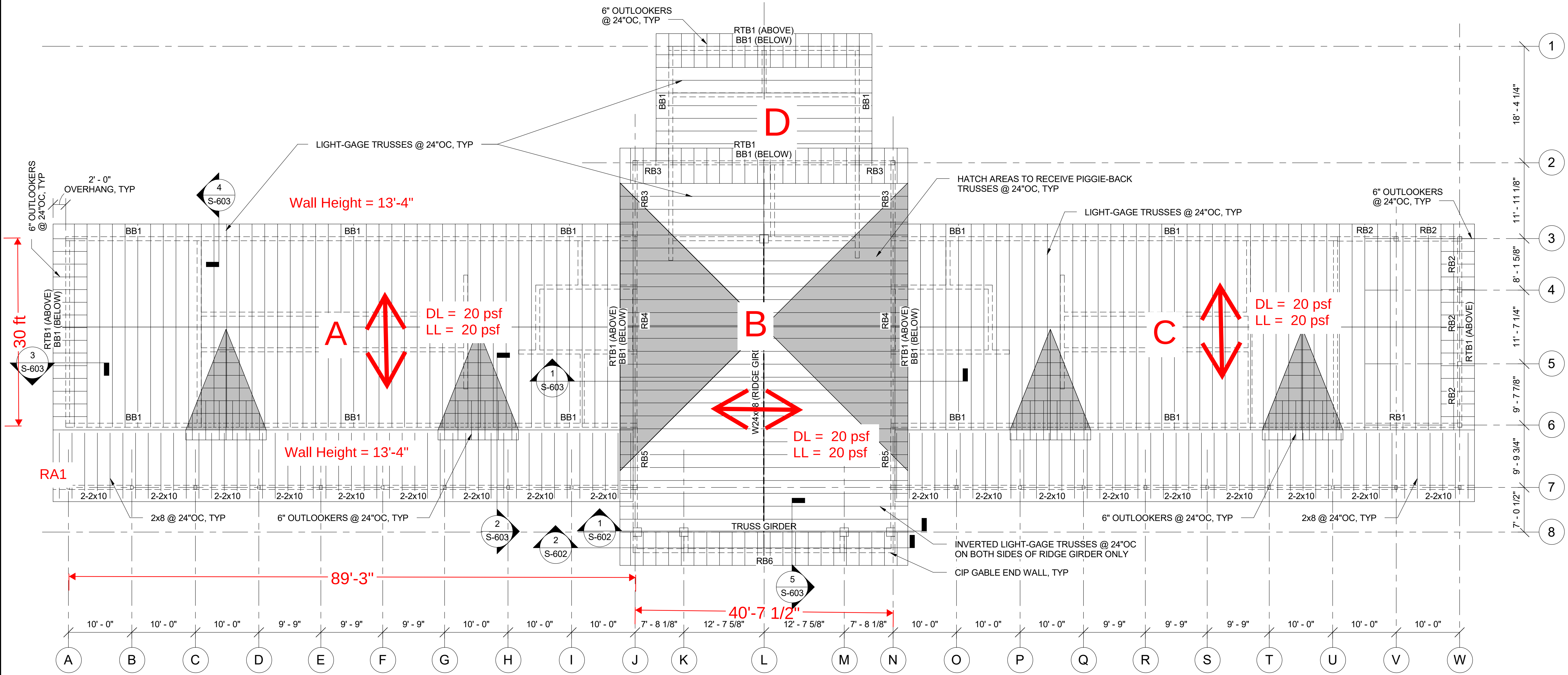
BUILDING SECTIONS

DRAWING NO.
A301

SHEET NO.
X OF XX

BEAM SCHEDULE

TYPE	SIZE	REINFORCEMENT		COMMENTS
		LONGITUDINAL	TIES	
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EE B/L@12"OC	
RB3	8x16	2#6 T&B	#3@6"OC	
RB4	8x16	2#6 T&B	#3@6"OC	
RB5	8x16	2#6 T&B	#3@6"OC	
RB6	8x48	2#6 T&B	#3@12"OC	
RTB1	8x12 (NOM)	2#5 T&B	#3@12"OC	RAKED TIE BEAM
BB1	2-COURSE BOND BEAM			



1 S-102 ROOF FRAMING PLAN
SCALE: 1/16" = 1'-0"

C:\Users\RUBI3304\AppData\Local\Temp\RVT2016\FDOT I10 Rest Areas R16_RUBI3304.rvt

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.
482 KELLER ROAD
ORLANDO, FLORIDA 32810
(407) 647-7275
CERTIFICATE OF AUTHORIZATION NO. 24
DOUGLAS A. RAMIREZ , P.E. NO. 70993

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

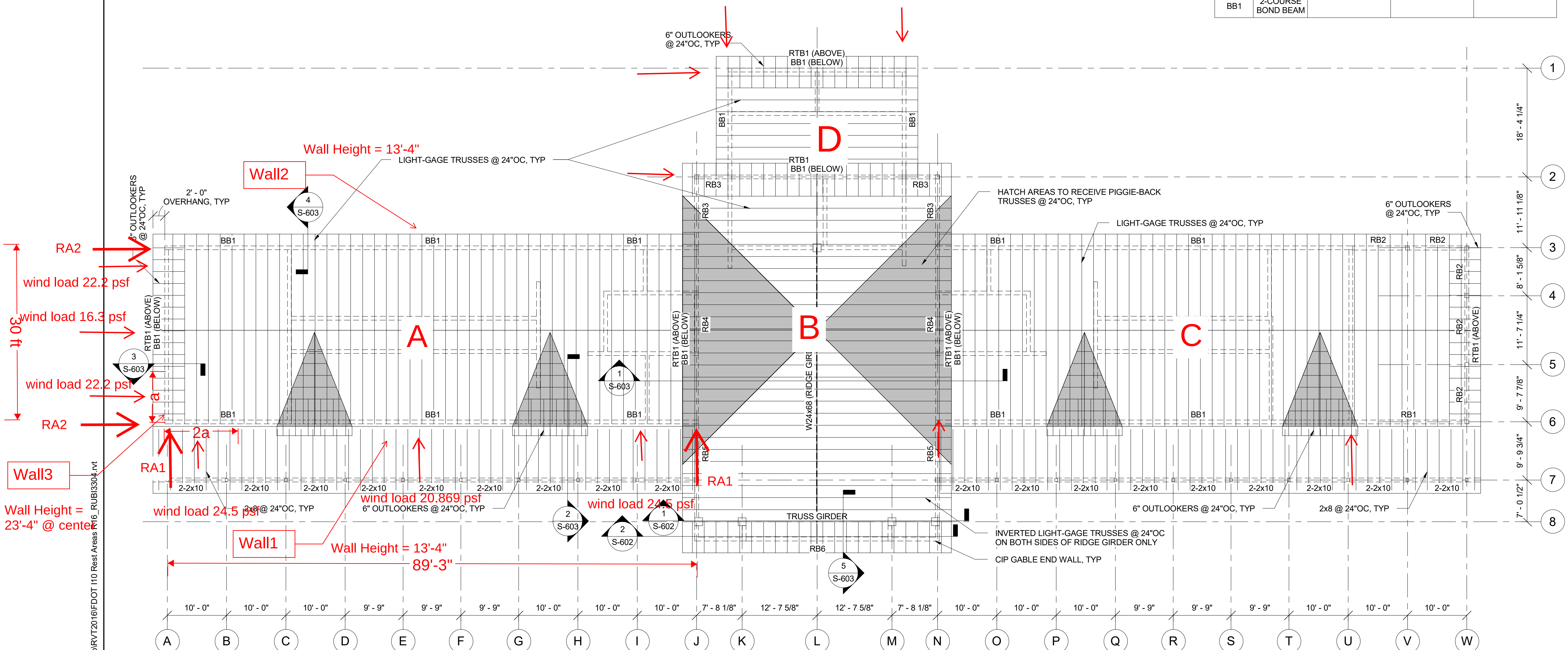
ROOF FRAMING PLAN

DRAWING NO. S-102

SHEET NO. XX OF XX

Rest Room Building Roof Deck

BEAM SCHEDULE				
TYPE	SIZE	REINFORCEMENT		COMMENTS
		LONGITUDINAL	TIES	
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EE B/L@12"OC	
RB3	8x16	2#6 T&B	#3@6"OC	
RB4	8x16	2#6 T&B	#3@6"OC	
RB5	8x16	2#6 T&B	#3@6"OC	
RB6	8x48	2#6 T&B	#3@12"OC	
RTB1	8x12 (NOM)	2#5 T&B	#3@12"OC	RAKED TIE BEAM
BB1	2-COURSE BOND BEAM			



Roof loading:

1
S-102
ROOF FRAMING PLAN
SCALE: 1/16" = 1'-0"

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REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.
482 KELLER ROAD
ORLANDO, FLORIDA 32810
(407) 647-7275
CERTIFICATE OF AUTHORIZATION NO. 24
DOUGLAS A. RAMIREZ , P.E. NO. 70993

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

ROOF FRAMING PLAN

DRAWING NO.
S-102
SHEET NO.
XX OF XX

DIAPHRAGM - 1.5B / 1.5F / 1.5A

use #12 TEK screws
per the spec

1.5B / 1.5F / 1.5A ROOF DECK

- **20 Gage**
- **Support Fasteners: 5/8” puddle welds[1]**
- **Sidelap Fasteners: #10 TEK screws**

Truss A loads : 16.8 kip over 30 ft = 560 plf

Truss B loads : 21.8 kip over 25.5 ft = 855 plf

36/7 Attachment Pattern

Factor of safety = 2.35

ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) — SPAN LENGTH (ft-in.)

Number of Sidelap Fasteners	3’-0”	3’-6”	4’-0”	4’-6”	5’-0”	5’-6”	6’-0”	6’-6”	7’-0”	7’-6”	8’-0”	8’-6”	9’-0”	9’-6”	10’-0”	K1
0	581	505	441	390	349											0.535
1	670	585	519	463	414	375	342									0.415
2	754	662	589	530	480	434	397	365	337	313	293	274	258			0.340
3	832	735	656	592	538	493	451	415	384	357	333	313	294	279	265	0.287
4	904	803	720	652	594	545	504	465	431	401	374	351	331	310	279	0.249
5	971	867	781	709	648	596	551	512	477	444	415	387	345	310	279	0.219
6	1032	927	839	764	700	645	597	556	520	488	437	387	345	310	279	0.196
7	1089	983	893	816	749	692	642	599	560	497	437	387	345	310	279	0.178
8	1140	1036	944	866	797	738	686	640	570	497	437	387	345	310	279	0.162
9	1187	1084	993	913	843	782	728	661	570	497	437	387	345	310	279	0.149
10	1230	1129	1038	957	886	824	769	661	570	497	437	387	345	310	279	0.138

D_F = 169 D_A = 266 D_B = 97 K2 = 1056

Truss over Center pavilion

36/5 Attachment Pattern

ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) — SPAN LENGTH (ft-in.)

Number of Sidelap Fasteners	3’-0”	3’-6”	4’-0”	4’-6”	5’-0”	5’-6”	6’-0”	6’-6”	7’-0”	7’-6”	8’-0”	8’-6”	9’-0”	9’-6”	10’-0”	K1
0	514	452	402	361	323											0.642
1	589	523	468	423	386	351	320									0.477
2	656	587	529	481	439	404	374	344	318	296	276	259	243			0.380
3	715	645	585	534	490	452	420	391	365	339	317	297	280	265	251	0.315
4	767	697	637	584	538	498	463	433	405	381	358	336	316	299	279	0.270
5	812	744	683	630	583	541	505	472	443	418	394	374	345	310	279	0.236
6	852	786	726	672	624	582	544	510	480	453	428	387	345	310	279	0.209
7	886	823	764	711	663	620	581	546	515	486	437	387	345	310	279	0.188
8	916	856	799	747	699	656	616	580	548	497	437	387	345	310	279	0.171
9	942	885	830	779	732	689	649	613	570	497	437	387	345	310	279	0.156
10	965	911	858	809	763	720	680	644	570	497	437	387	345	310	279	0.144

D_F = 663 D_A = 728 D_B = 567 K2 = 1056

36/4 Attachment Pattern

ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) — SPAN LENGTH (ft-in.)

Number of Sidelap Fasteners	3’-0”	3’-6”	4’-0”	4’-6”	5’-0”	5’-6”	6’-0”	6’-6”	7’-0”	7’-6”	8’-0”	8’-6”	9’-0”	9’-6”	10’-0”	K1
0	393	346	309	273	244											0.802
1	466	415	373	338	308	279	254									0.561
2	528	476	431	393	360	332	308	284	262	243	227	212	199			0.431
3	580	528	482	443	408	378	352	328	308	287	268	251	236	223	212	0.350
4	623	573	528	488	452	420	392	368	345	326	308	289	272	258	245	0.294
5	659	612	568	528	492	459	430	404	381	360	341	323	308	292	277	0.254
6	689	645	603	563	528	495	466	439	414	392	372	354	337	310	279	0.224
7	715	673	633	595	560	528	498	471	446	423	402	383	345	310	279	0.200
8	736	697	659	623	589	557	528	500	475	452	430	387	345	310	279	0.180
9	753	718	682	648	615	584	555	527	502	479	437	387	345	310	279	0.164
10	768	736	702	670	638	608	580	553	527	497	437	387	345	310	279	0.151

D_F = 909 D_A = 959 D_B = 802 K2 = 1056

[1] A 3/8” x 1-1/4” arc seam weld shall be used with F deck or A Deck.

$$G' = \frac{K_2}{3.78 + \frac{0.3 \cdot D_x}{SPAN} + 3 \cdot K_1 \cdot SPAN}, \text{ Kips/inch}$$

SPAN is in feet Substitute D_B, D_F, or D_A for D_x

Truss over Restroom
and storage area

NUCOR VULCRAFT/VERCO GROUP
Better Partners. Better Products.
Better Outcomes.

1.5B / 1.5F / 1.5A ROOF DECK

- **20 Gage**
- **Support Fasteners: #12 TEK screws**
- **Sidelap Fasteners: #10 TEK screws**

Truss A loads : 16.8 kip over 30 ft = 560 plf

Truss B loads : 21.8 kip over 25.5 ft = 855 plf

36/7 Attachment Pattern **Factor of safety = 2.35**

Number of Sidelap Fasteners	ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) SPAN LENGTH (ft-in.) 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	K1
0	343	298	261	230	206											0.605
1	430	377	335	301	272	246	224									0.456
2	508	450	402	363	331	303	279	257	237	221	206	193	182			0.366
3	576	515	464	421	385	354	328	305	284	264	247	232	218	207	196	0.306
4	636	573	520	475	436	402	373	348	325	306	288	270	255	241	229	0.263
5	687	625	571	524	483	448	417	389	365	343	324	306	291	276	262	0.230
6	730	671	617	569	527	490	458	428	402	379	358	340	323	307	279	0.205
7	768	710	658	611	568	530	496	466	438	414	392	372	345	310	279	0.185
8	800	745	694	648	605	567	532	501	473	447	424	387	345	310	279	0.168
9	827	776	727	681	639	601	566	534	505	478	437	387	345	310	279	0.154
10	851	803	756	712	671	632	597	565	536	497	437	387	345	310	279	0.142

D_F = 169 D_A = 266 D_B = 97 K2 = 1056

36/5 Attachment Pattern

Number of Sidelap Fasteners	ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) SPAN LENGTH (ft-in.) 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	K1
0	304	267	238	213	191											0.726
1	376	335	302	274	250	230	211									0.522
2	435	393	358	327	301	278	258	241	225	210	197	184	173			0.408
3	482	441	406	374	346	322	300	281	264	248	235	222	210	199	189	0.334
4	519	481	446	415	387	361	338	318	299	283	268	254	242	230	220	0.283
5	548	513	481	450	422	396	373	352	333	315	299	284	271	259	247	0.246
6	572	540	509	480	453	428	404	383	363	345	328	313	299	286	274	0.217
7	590	562	534	506	480	455	432	411	391	372	355	339	325	310	279	0.195
8	606	580	554	528	504	480	457	436	416	398	380	364	345	310	279	0.176
9	618	595	571	547	524	501	480	459	440	421	404	387	345	310	279	0.161
10	628	607	586	564	542	520	500	480	460	442	425	387	345	310	279	0.148

D_F = 663 D_A = 728 D_B = 567 K2 = 1056

36/4 Attachment Pattern

Number of Sidelap Fasteners	ALLOWABLE DIAPHRAGM SHEAR STRENGTH (plf) SPAN LENGTH (ft-in.) 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	K1
0	232	205	182	161	144											0.907
1	301	270	244	222	204	188	173									0.610
2	353	323	296	272	251	233	217	203	191	179	167	157	148			0.459
3	391	363	337	313	292	273	256	240	227	214	203	192	183	174	165	0.368
4	419	394	370	347	327	307	290	274	259	246	233	222	212	203	194	0.307
5	439	418	396	375	355	337	319	303	288	274	261	250	239	229	219	0.264
6	455	436	417	397	379	361	344	329	314	300	287	275	263	253	243	0.231
7	467	450	433	416	399	382	366	351	336	323	310	297	286	275	265	0.206
8	476	461	446	431	415	400	385	370	356	343	330	318	306	295	279	0.185
9	483	471	457	443	429	415	401	387	373	361	348	336	325	310	279	0.168
10	489	478	466	453	440	427	414	401	389	376	364	353	341	310	279	0.154

D_F = 909 D_A = 959 D_B = 802 K2 = 1056

$$G' = \frac{K_2}{3.78 + \frac{0.3 \cdot D_x}{SPAN} + 3 \cdot K_1 \cdot SPAN}, \text{ Kips/inch}$$

SPAN is in feet Substitute D_B, D_F, or D_A for D_x

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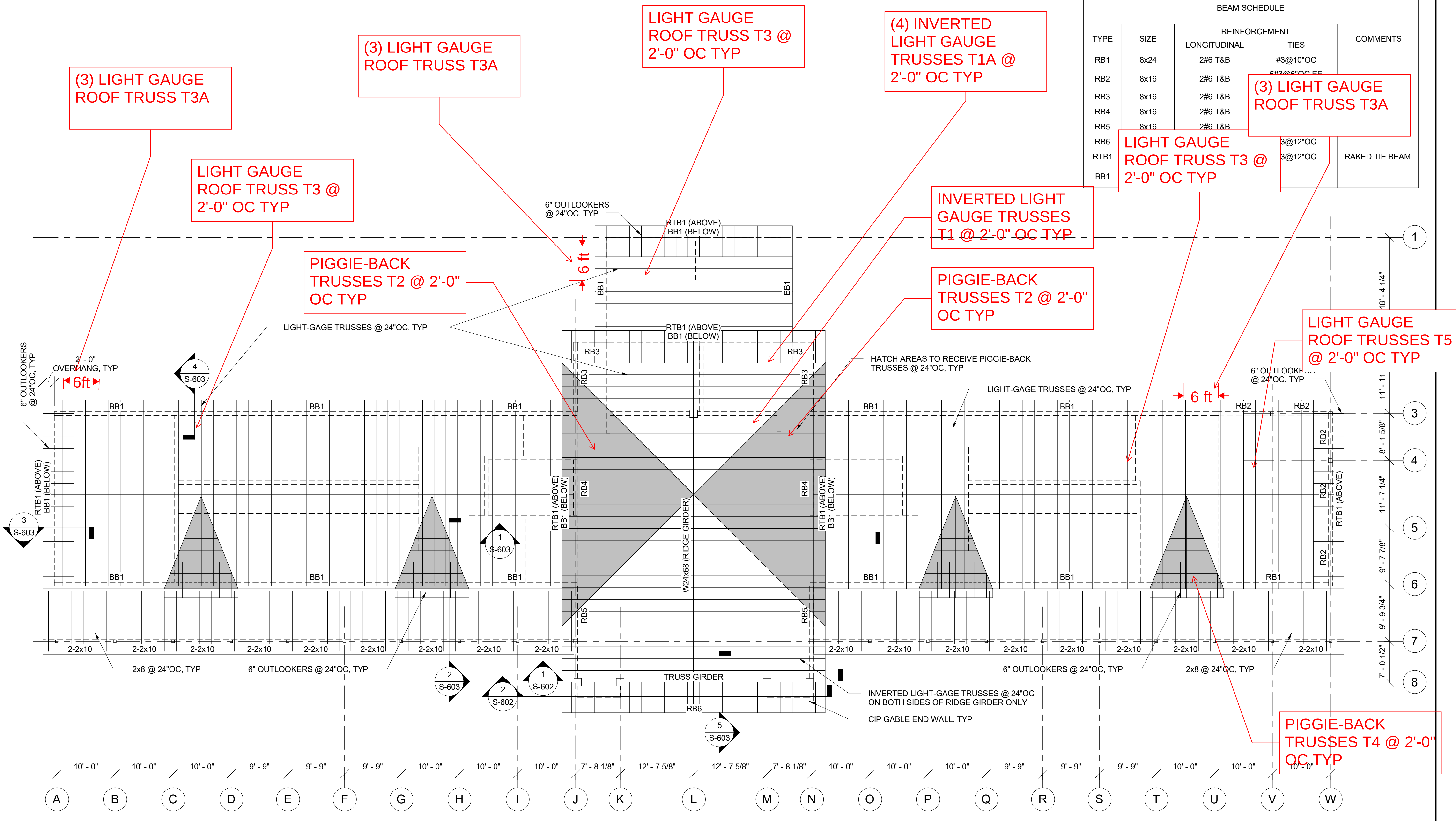
Better Outcomes.

Rest Room Building Roof Trusses

BEAM SCHEDULE

TYPE	SIZE	REINFORCEMENT		COMMENTS
		LONGITUDINAL	TIES	
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EF	
RB3	8x16	2#6 T&B	[illegible]	[illegible]
RB4	8x16	2#6 T&B	[illegible]	[illegible]
RB5	8x16	2#6 T&B	[illegible]	[illegible]
RB6	[illegible]	[illegible]	3@12"OC	
RTB1	[illegible]	[illegible]	3@12"OC	RAKED TIE BEAM
BB1	[illegible]	[illegible]		



1 S-102 ROOF FRAMING PLAN

SCALE: 1/16" = 1'-0"

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REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.
482 KELLER ROAD
ORLANDO, FLORIDA 32810
(407) 647-7275
CERTIFICATE OF AUTHORIZATION NO. 24
DOUGLAS A. RAMIREZ , P.E. NO. 70993

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

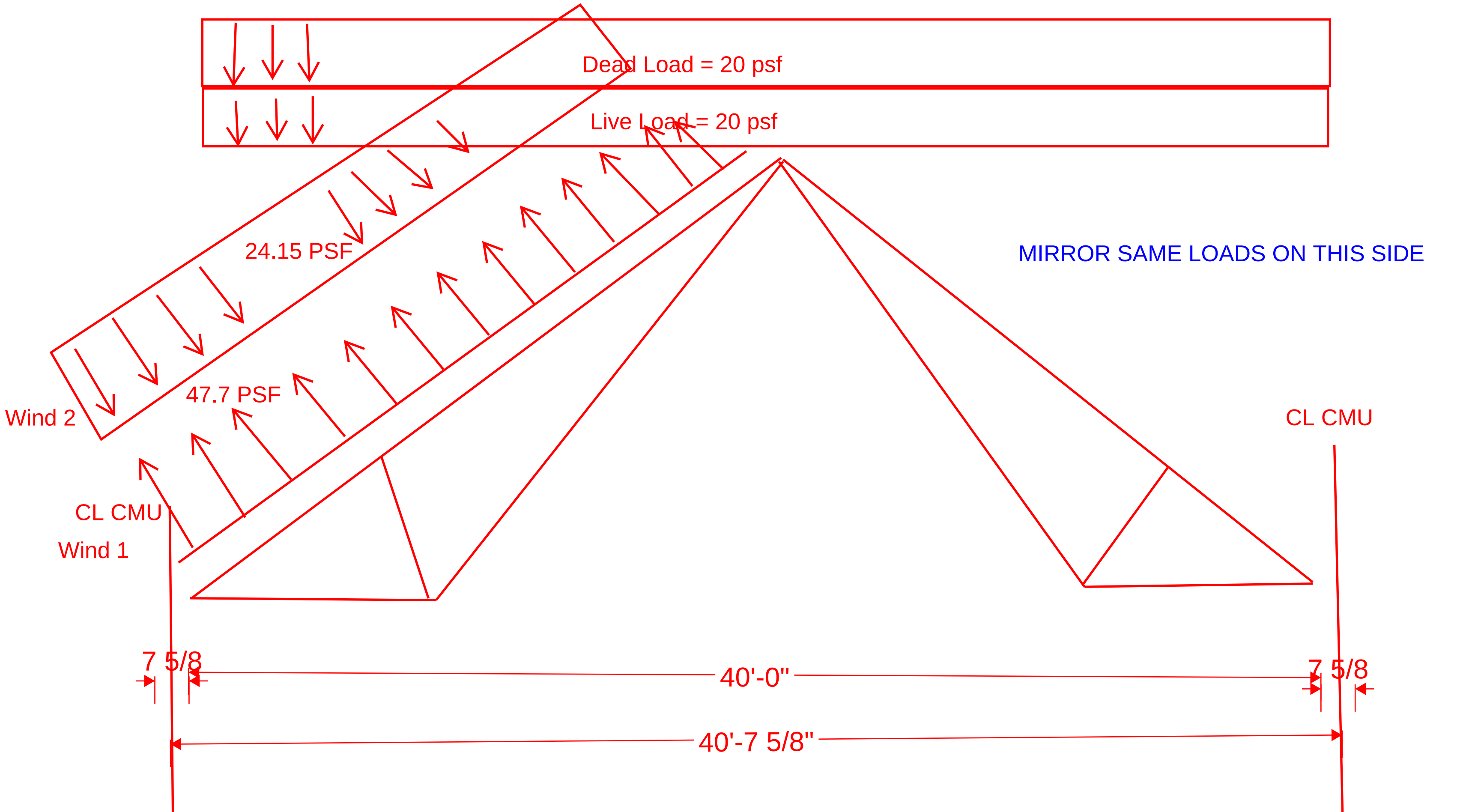
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

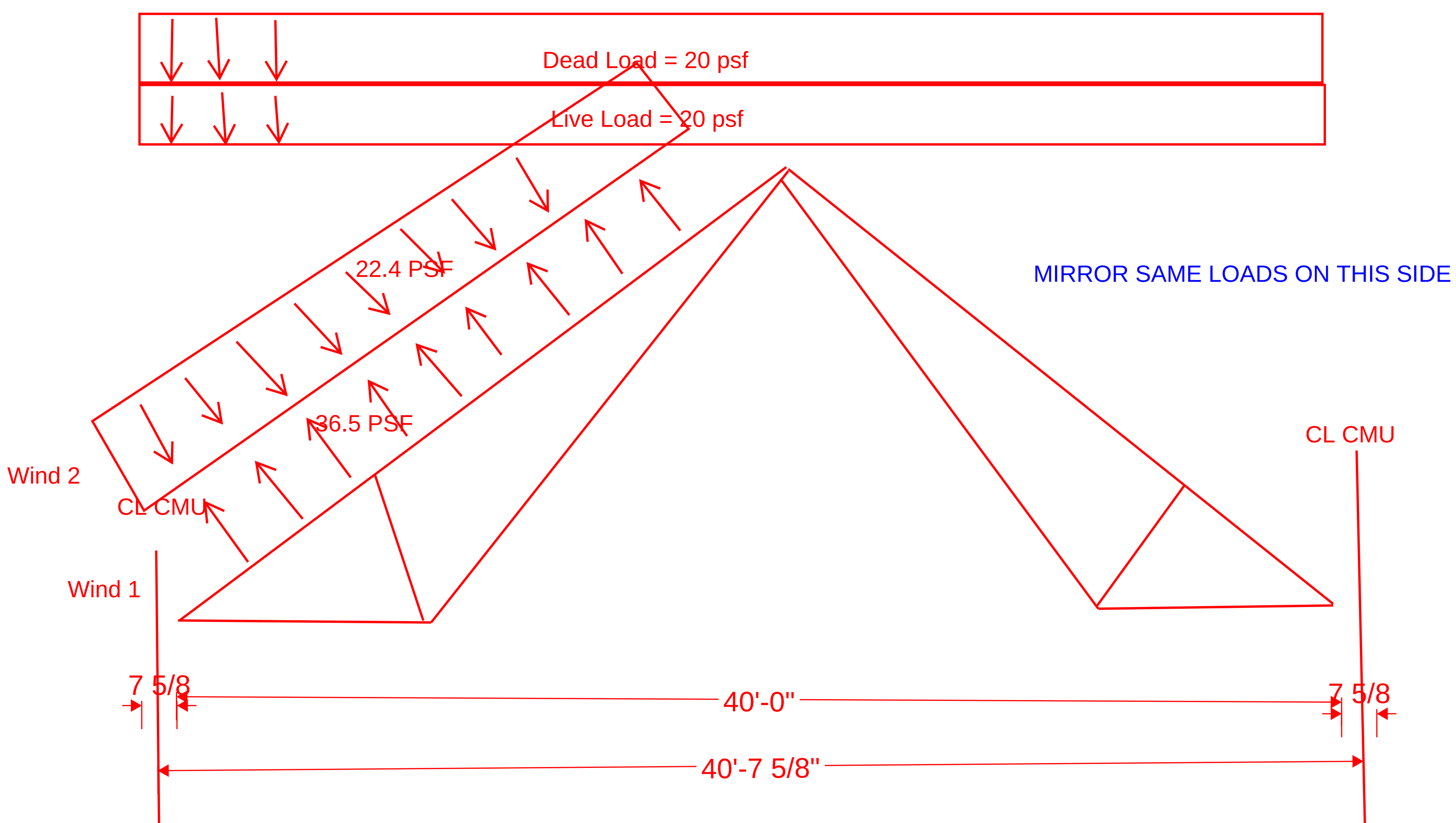
ROOF FRAMING PLAN

DRAWING NO. S-102

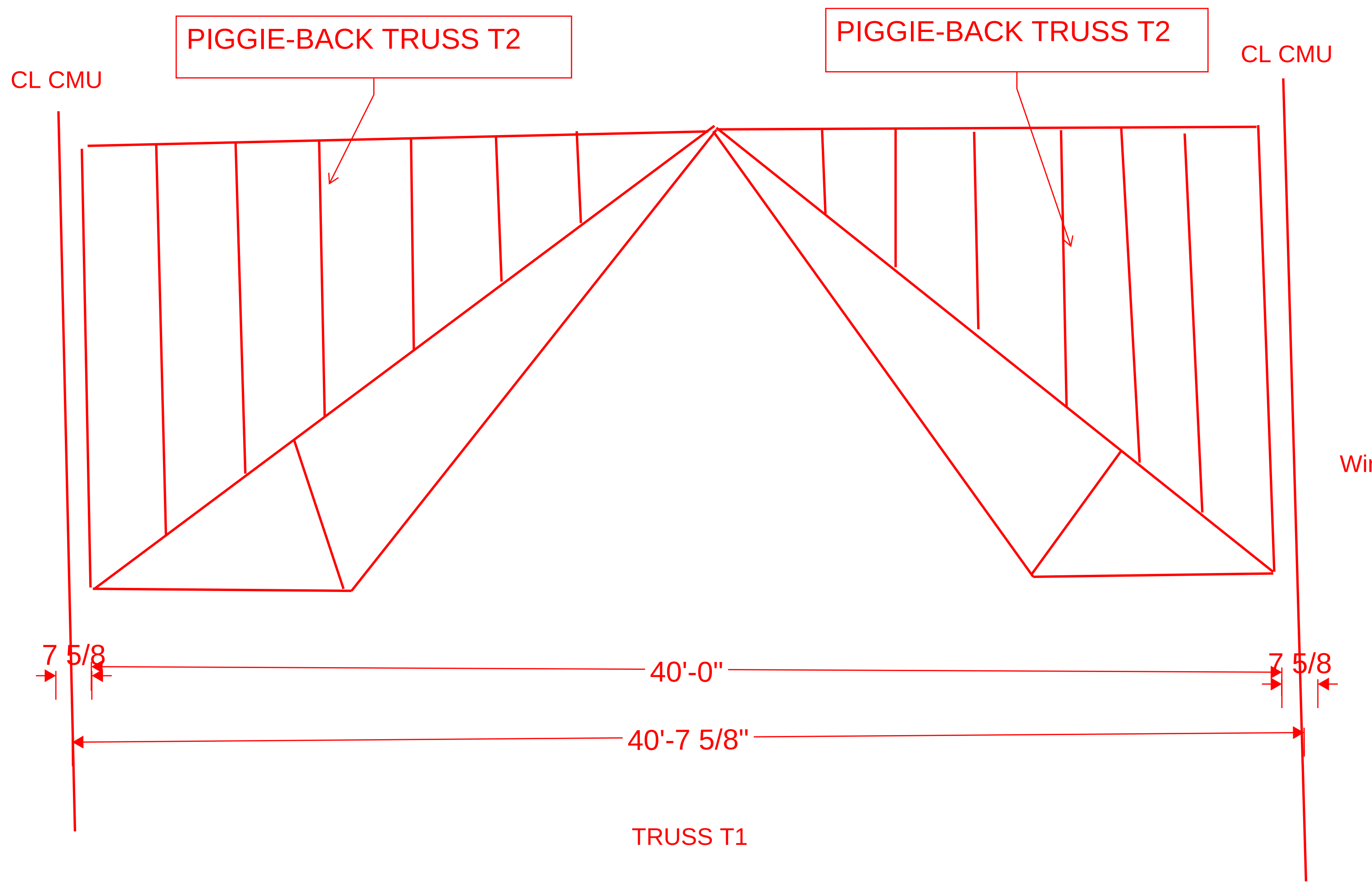
SHEET NO. XX OF XX



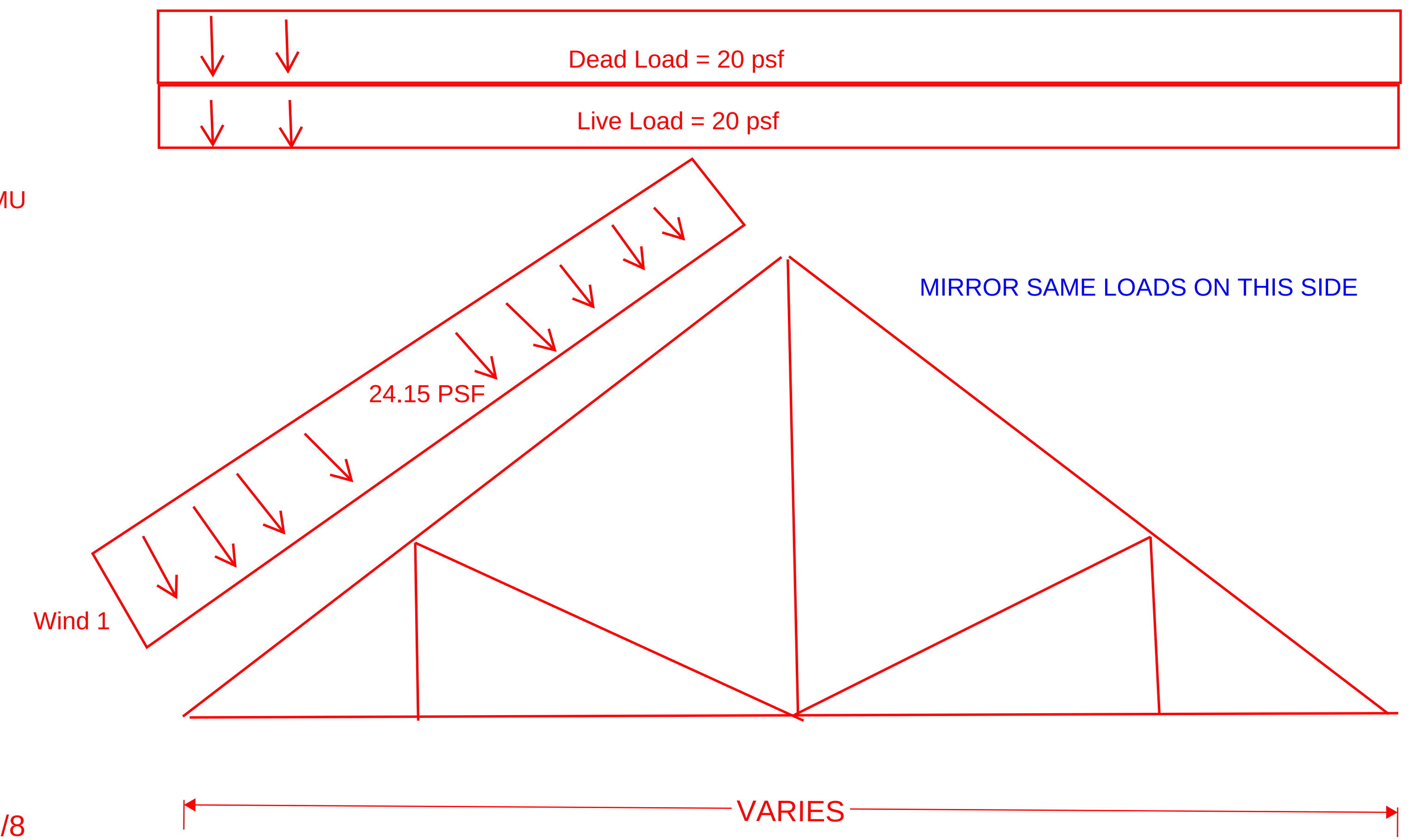
TRUSS T1A



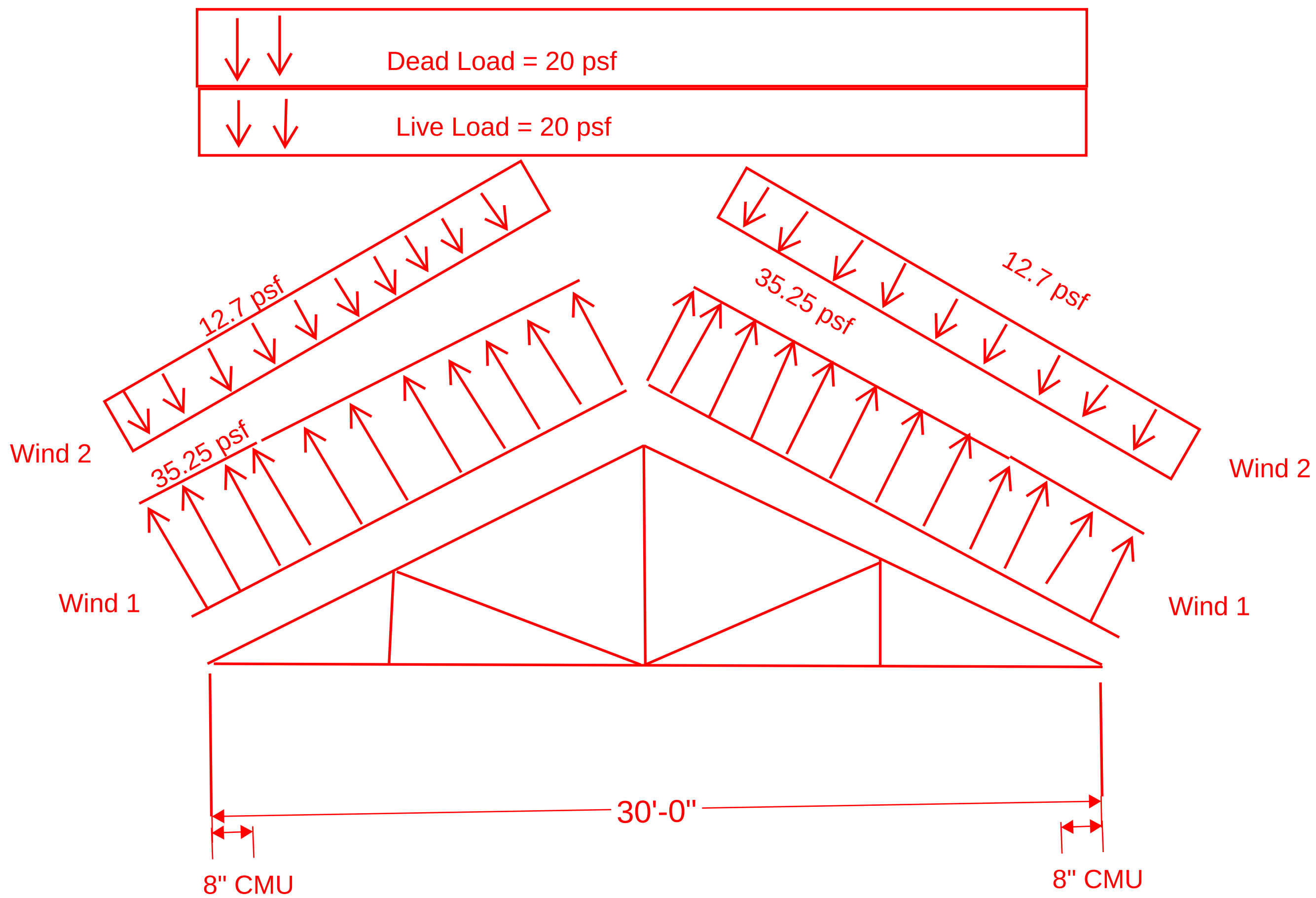
TRUSS T1



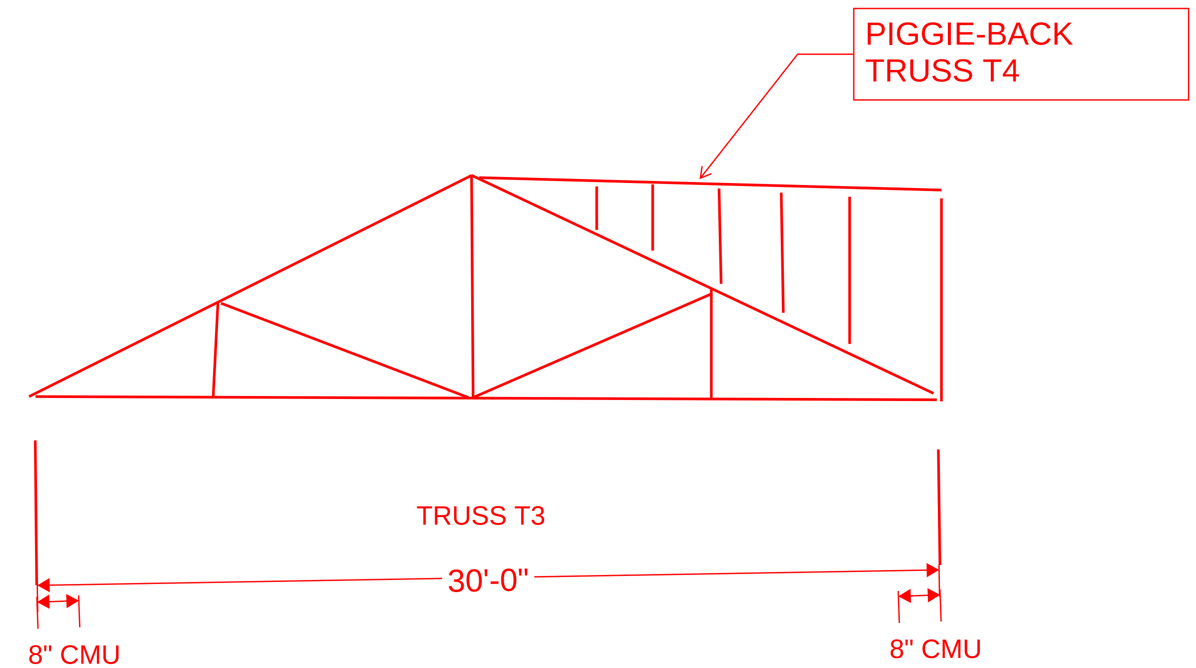
TRUSS T1

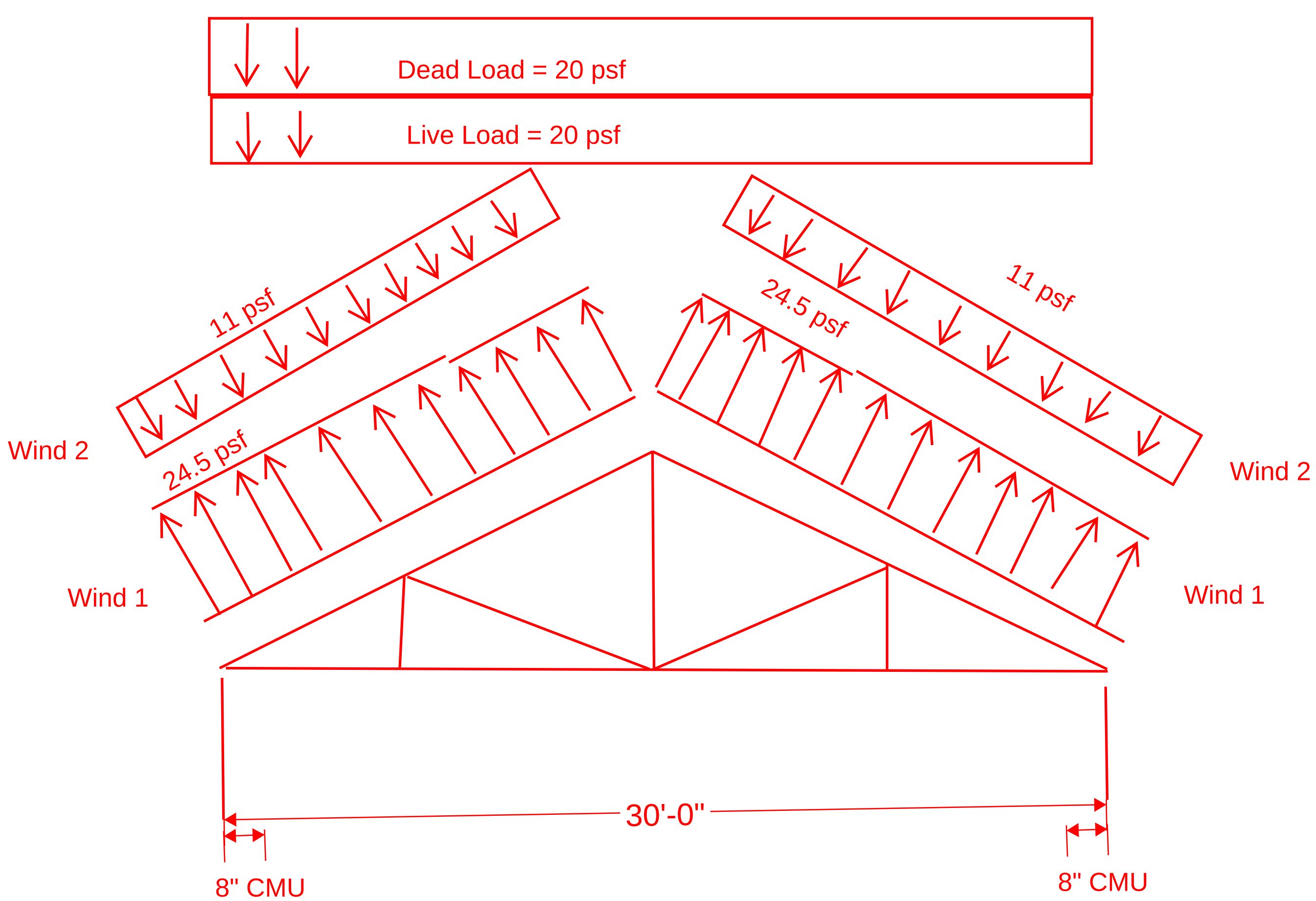


TRUSS T2

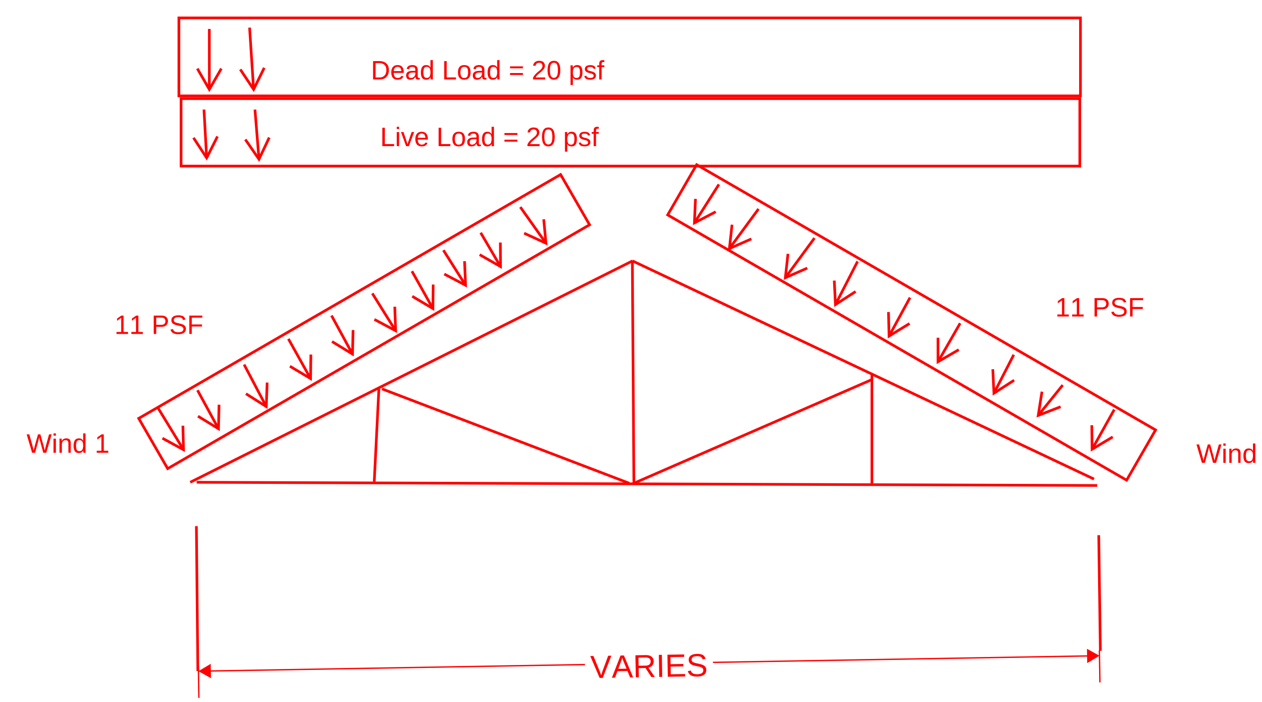


TRUSS T3A

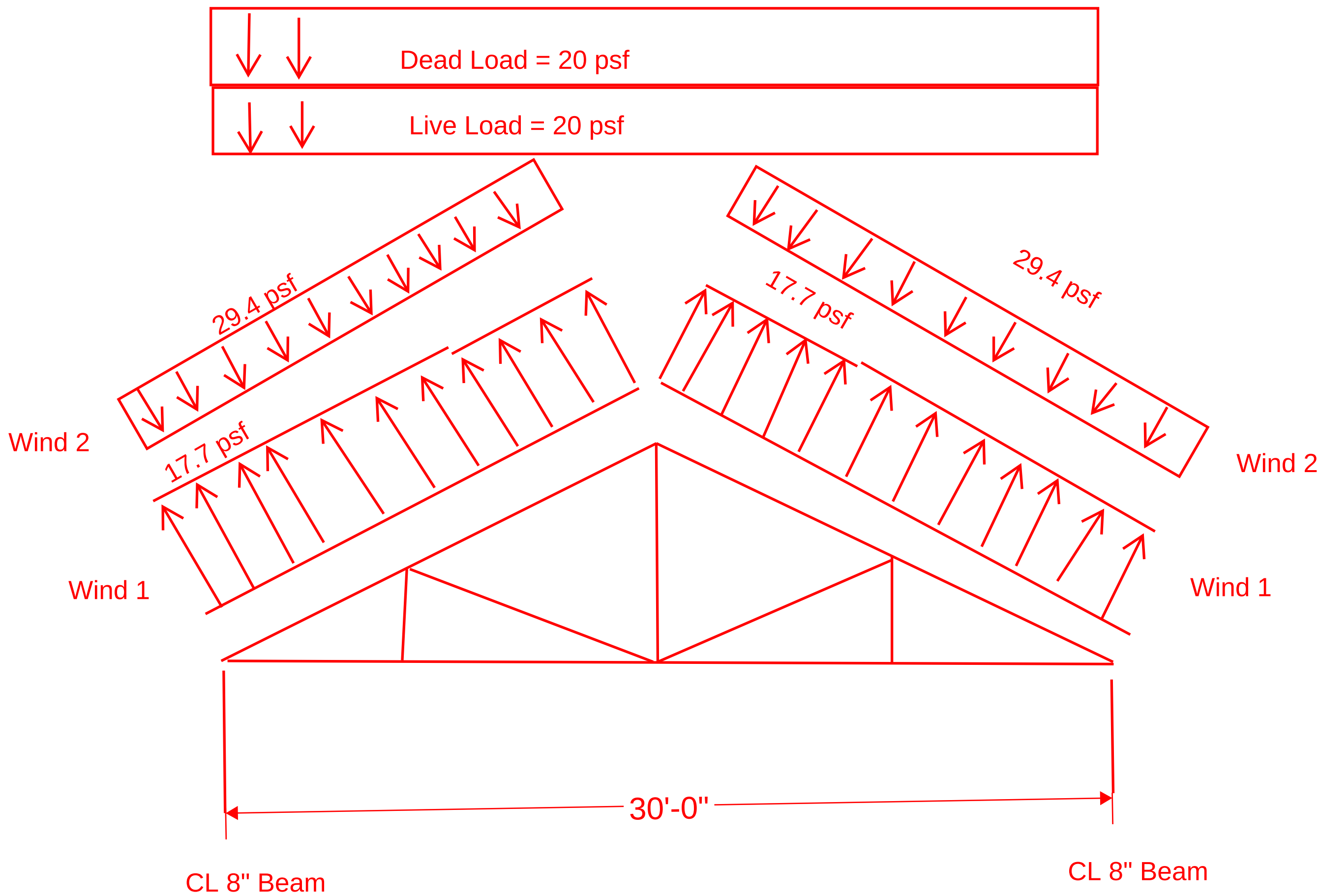




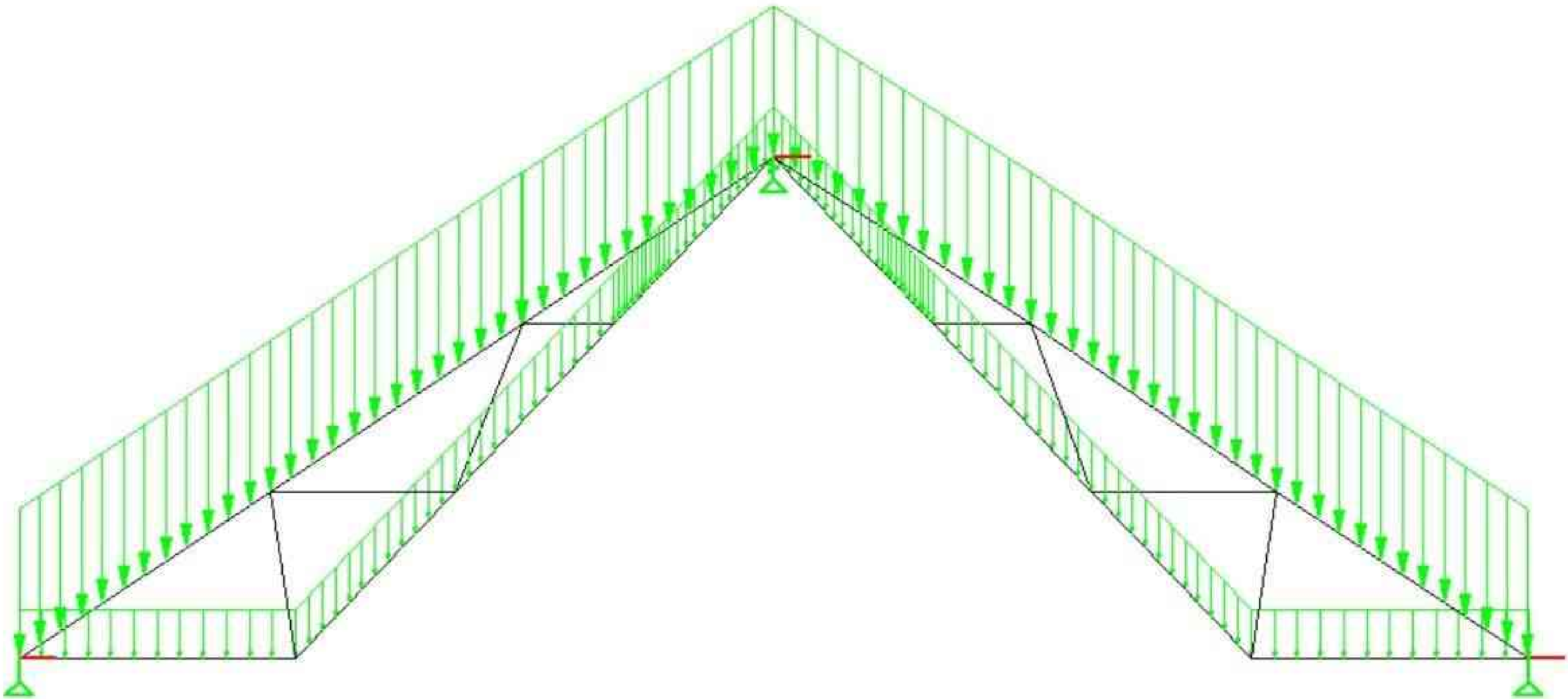
TRUSS T3



TRUSS T4

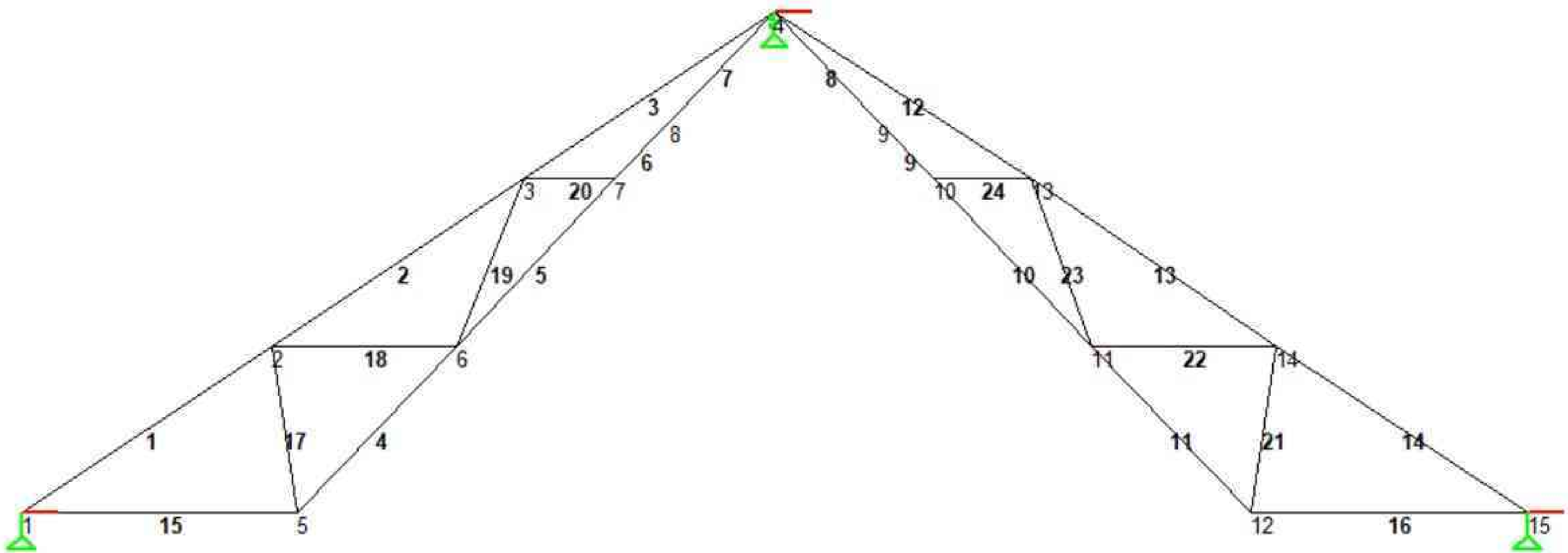

Dead Load = 20 psf
Live Load = 20 psf
29.4 psf
17.7 psf
Wind 2
Wind 1
17.7 psf
29.4 psf
Wind 2
Wind 1
30'-0"
CL 8" Beam
CL 8" Beam

TRUSS T5



TRUSS LOADING (DL)

LL IS SIMILAR (T.C. ONLY)



TRUSS JOINTS AND MEMBERS

TRUSS REACTIONS

SUPPORT REACTIONS -UNIT POUN FEET STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	307.92	463.24	0.00	0.00	0.00	0.00
	2	543.35	827.71	0.00	0.00	0.00	0.00
	3	524.44	816.30	0.00	0.00	0.00	0.00
	4	-466.38	-1063.30	0.00	0.00	0.00	0.00
	5	388.02	348.26	0.00	0.00	0.00	0.00
	100	1375.72	2107.25	0.00	0.00	0.00	0.00
	101	571.45	652.97	0.00	0.00	0.00	0.00
	102	1084.09	1499.91	0.00	0.00	0.00	0.00
	103	1034.74	1424.69	0.00	0.00	0.00	0.00
	104	1419.22	2059.89	0.00	0.00	0.00	0.00
	105	230.94	136.59	0.00	0.00	0.00	0.00
	106	743.58	983.53	0.00	0.00	0.00	0.00
15	1	-308.44	464.20	0.00	0.00	0.00	0.00
	2	-544.21	828.90	0.00	0.00	0.00	0.00
	3	-525.26	817.68	0.00	0.00	0.00	0.00
	4	254.52	-1165.45	0.00	0.00	0.00	0.00
	5	88.72	574.99	0.00	0.00	0.00	0.00
	100	-1377.91	2110.78	0.00	0.00	0.00	0.00
	101	-699.94	593.84	0.00	0.00	0.00	0.00
	102	-799.42	1638.10	0.00	0.00	0.00	0.00
	103	-1132.06	1381.91	0.00	0.00	0.00	0.00
	104	-1206.67	2165.11	0.00	0.00	0.00	0.00
	105	-358.88	76.59	0.00	0.00	0.00	0.00
	106	-458.36	1120.86	0.00	0.00	0.00	0.00
4	1	0.52	173.44	0.00	0.00	0.00	0.00
	2	0.86	300.10	0.00	0.00	0.00	0.00
	3	0.81	289.01	0.00	0.00	0.00	0.00
	4	-12.81	-276.10	0.00	0.00	0.00	0.00
	5	25.91	113.54	0.00	0.00	0.00	0.00
	100	2.19	762.55	0.00	0.00	0.00	0.00
	101	-6.31	307.88	0.00	0.00	0.00	0.00
	102	16.92	541.67	0.00	0.00	0.00	0.00
	103	-3.77	566.06	0.00	0.00	0.00	0.00
	104	13.65	741.40	0.00	0.00	0.00	0.00
	105	-6.86	118.47	0.00	0.00	0.00	0.00
	106	16.37	352.25	0.00	0.00	0.00	0.00

1. DL - SELFWEIGHT
2. DL - EXTERNAL
3. LL_r
4.WL - NEGATIVE
5.WL - POSITIVE

100. LC2 -DL+LL_r
101. LC5 -DL+0.6WL NEG
102. LC5 - DL+0.6WL POS
103. LC6a -DL+0.75LL_r+0.45WL NEG
104. LC6a -DL+0.75LL_r+0.45WL POS
106. LC7 -0.6DL+0,6WL NEG
107. LC7 -O.6DL+0.6WL POS

Rest Room Building Central Area Beams

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318

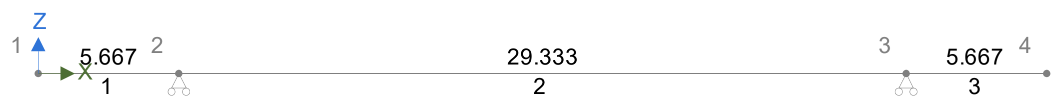
Tedds calculation version 3.3.03

ANALYSIS

Tedds calculation version 1.0.31

Geometry

Geometry (ft) - Concrete (4000 150) - R 8x24

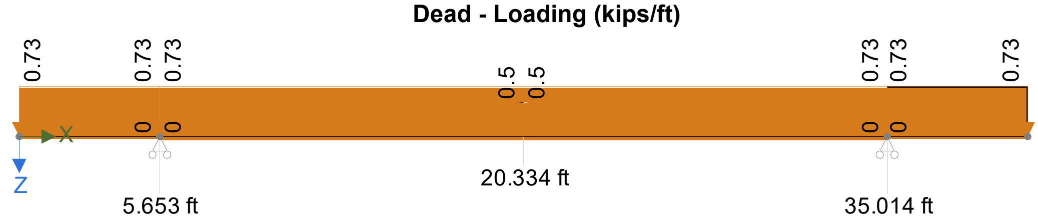


Span	Length (ft)	Section	Start Support	End Support
1	5.67	R 8x24	Free	Roller Pin X
2	29.33	R 8x24	Roller Pin X	Roller Pin X
3	5.67	R 8x24	Roller Pin X	Free
R 8x24: Area 192 in^2, Inertia Major 9216 in^4, Inertia Minor 1024 in^4, Shear area parallel to Minor 160 in^2, Shear area parallel to Major 160 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead
1.4D (Strength)	1.40	1.40
1.2D+1.6L (Strength)	1.20	1.20

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.73 kips/ft
Beam	Dead	UDL	GlobalZ	0 kips/ft
Beam	Dead	VDL	GlobalZ	0 kips/ft at 5.653 ft to 0.5 kips/ft at 20.334 ft
Beam	Dead	VDL	GlobalZ	0.5 kips/ft at 20.334 ft to 0 kips/ft at 35.014 ft

Results

Total deflection

Member results

Load combination: 1.4D (Strength)

Member	Position (ft)	Deflection (in)		Axial deflection (in)	
Beam	0	-0.41 (min)		0	
	20.33	0.72 (max)		0	
	40.67	-0.41 (min)		0	

Member results

Load combination: 1.2D+1.6L (Strength)

Member	Position (ft)	Deflection (in)		Axial deflection (in)	
Beam	0	-0.35 (min)		0	
	20.33	0.62 (max)		0	
	40.67	-0.35 (min)		0	

Total base reactions

Load case/combination	Force	
	FX (kips)	FZ (kips)
1.4D (Strength)	0	62.957
1.2D+1.6L (Strength)	0	53.963

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	4.067	0
3	0	4.067	0

Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	18.418	0
3	0	18.418	0

Load combination: 1.4D (Strength)

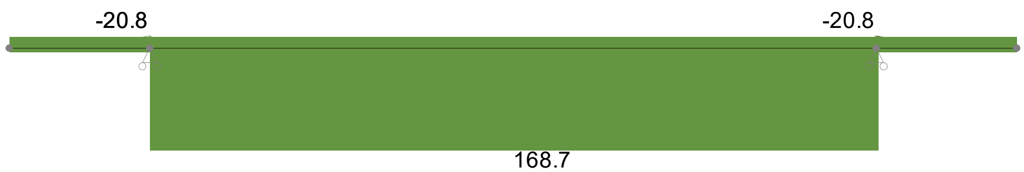
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	31.479	0
3	0	31.479	0

Load combination: 1.2D+1.6L (Strength)

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	26.982	0
3	0	26.982	0

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Concrete details

Compressive strength of concrete f'_c = **4000** psi

Density of reinforced concrete w_c = **150** lb / ft^3

Concrete type Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1) $E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi

Strength reduction factor for shear ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Nominal cover to reinforcement

Cover to top reinforcement c_{nom_t} = **1.5** in

Cover to bottom reinforcement c_{nom_b} = **1.5** in

Cover to side reinforcement c_{nom_s} = **1.5** in

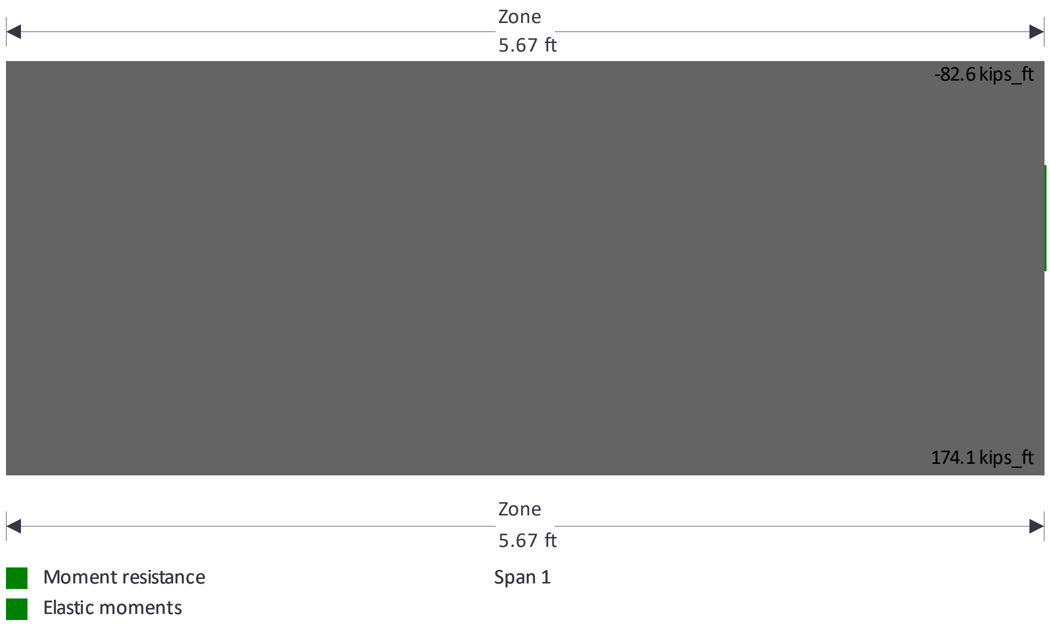
Beam - Span 1

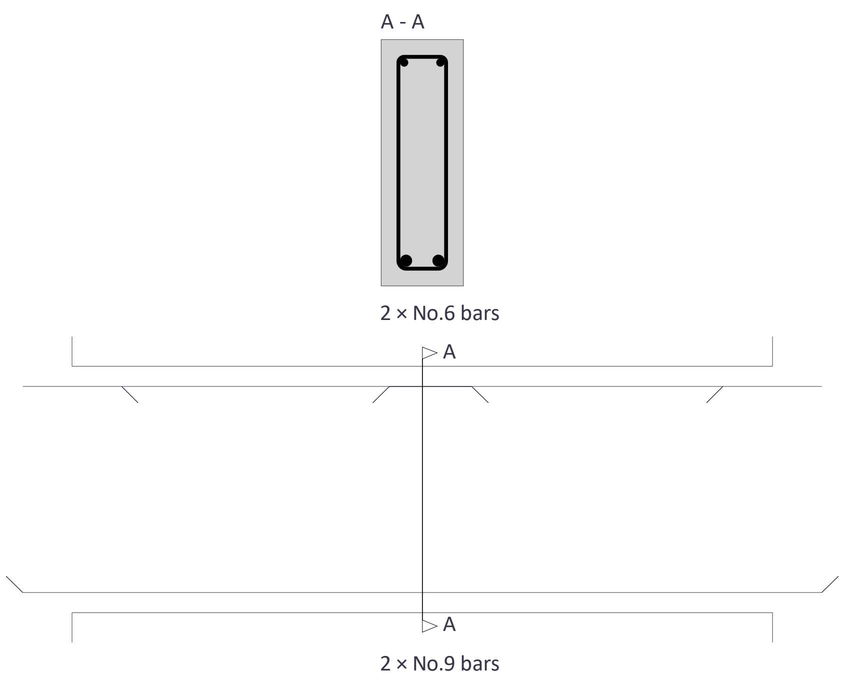
Rectangular section details

Section width b = **8** in

Section depth h = **24** in

Moment design





Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s1\_z2\_min\_red}$) = **20.801** kip_ft
Effective depth to tension reinforcement	d = **21.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × $\sqrt{(f'_c / 1 \text{ psi})}$, 200 psi) × b × d / f_y = **0.580** in^2

Stress block depth factor (cl.22.2.2.4.3) β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **1.949** in

Depth to neutral axis c = a / β_1 = **2.293** in

Net tensile strain in extreme tension fibers ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.02546**

Strength reduction factor (cl.21.2.1) ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**

Nominal moment strength M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **91.783** kip_ft

Design moment strength ϕM_n = M_n × ϕ_f = **82.605** kip_ft

Flexural cracking

Max. center to center spacing of tension reinf. $s_{t,max}$ = s_{top} + $\phi_{m1\_s1\_z2\_t\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement c_c = c_{nom_t} + ϕ_v = **1.875** in

Maximum allowable top bar spacing (Table 24.3.2) s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

Spacing limits for reinforcement

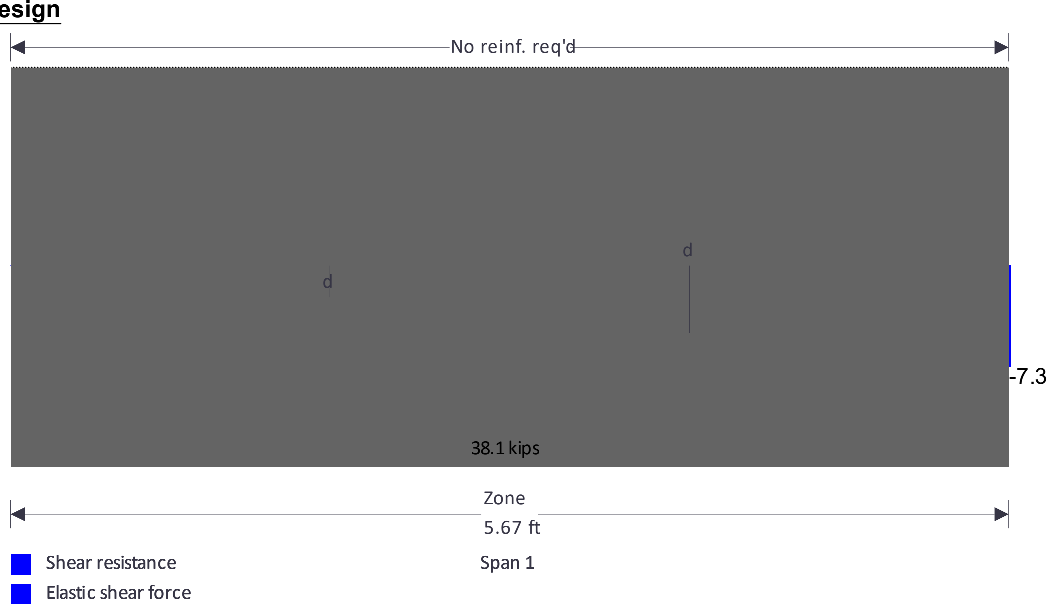
Top bar clear spacing s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_t\_L1}$ × $N_{m1\_s1\_z2\_t\_L1}$)) / ($N_{m1\_s1\_z2\_t\_L1}$ - 1) = **2.750** in

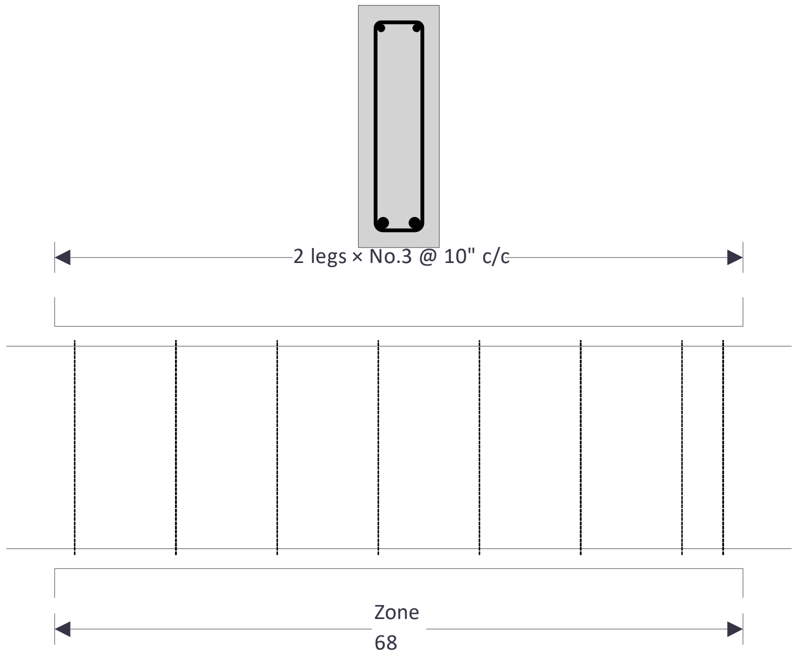
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_b\_L1}$ × $N_{m1\_s1\_z2\_b\_L1}$)) / ($N_{m1\_s1\_z2\_b\_L1}$ - 1) = **1.994** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

Shear design





Rectangular section in shear

Concrete weight modification factor λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less ϕV_c / 2) Between 0.00 ft and 5.67 ft

Maximum reinforcement shear strength $\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000psi) / 1\ psi)} \times b \times d$ = **66.028** kips

Minimum area of shear reinf. (Table 9.6.3.3) $A_{sv,min} = \max(50\ psi, 0.75\ psi \times \sqrt{(f'_c / 1\ psi)}) \times b / \min(f_y, 60000\ psi)$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone d = **21.75** in

Concrete shear strength (eqn. 22.5.5.1) $\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1\ psi)} \times b \times d$ = **16.507** kips

Design shear force within zone V_u = **4.993** kips

Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0\ kips)$ = **0.000** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d)$ = **0.000** in^2/ft

Area of shear reinforcement required $A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft

Shear reinforcement provided 2 legs × No.3 @ 10" c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **0.265** in^2/ft

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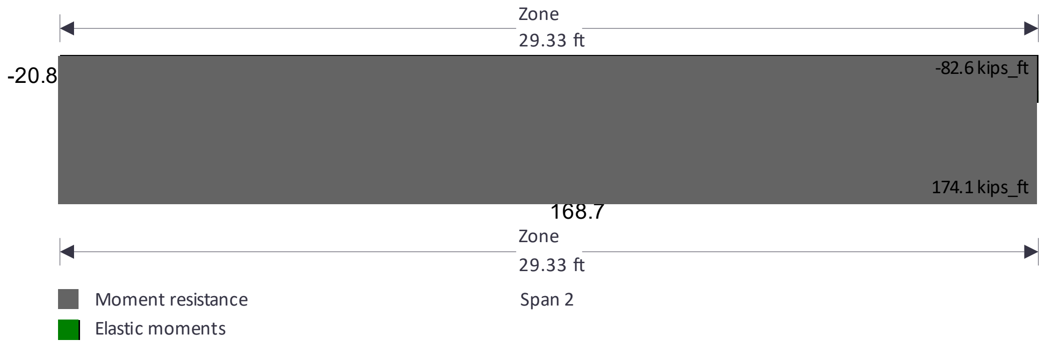
Beam - Span 2

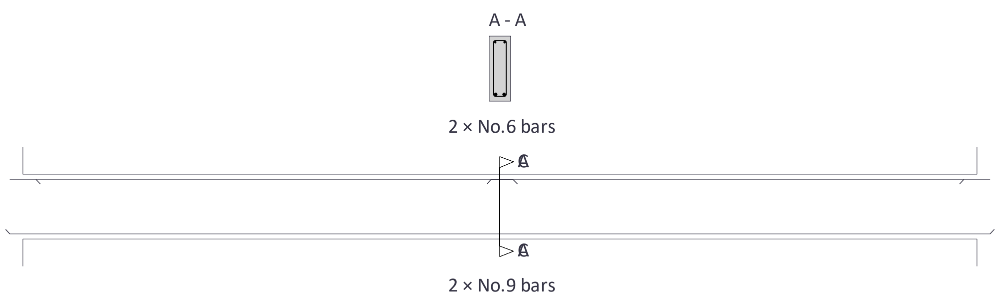
Rectangular section details

Section width b = **8** in

Section depth h = **24** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s2\_z2\_max\_red}$) = **168.741** kip_ft
Effective depth to tension reinforcement	d = **21.561** in
Tension reinforcement provided	2 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **1.999** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **0.575** in^2
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **4.409** in
Depth to neutral axis	c = a / β_1 = **5.187** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.00947**
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **193.436** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **174.092** kip_ft

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s2\_z2\_b\_L1}$ = **3.122** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **1.875** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y$ / 100000 psi = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s2} / 21.0) \times K_w \times K_f$ = **16.762** in

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Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s2\_z2\_min\_red})$ = **20.801** kip_ft
Effective depth to tension reinforcement	d = **21.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.580** in^2

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Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in
Depth to neutral axis	$c = a / \beta_1$ = **2.293** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.02546**

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Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **91.783** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **82.605** kip_ft

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Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max} = s_{top} + \phi_{m1\_s2\_z2\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_t} + \phi_v$ = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

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Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y$ / 100000 psi = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s2} / 21.0) \times K_w \times K_f$ = **16.762** in

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Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2\_t\_L1} \times N_{m1\_s2\_z2\_t\_L1})) / (N_{m1\_s2\_z2\_t\_L1} - 1)$ = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2\_b\_L1} \times N_{m1\_s2\_z2\_b\_L1})) / (N_{m1\_s2\_z2\_b\_L1} - 1)$ = **1.994** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

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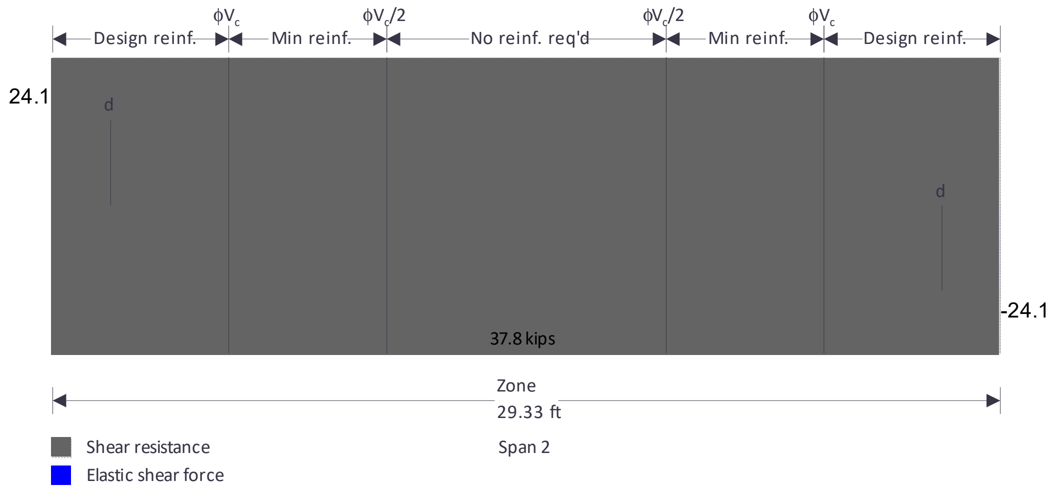
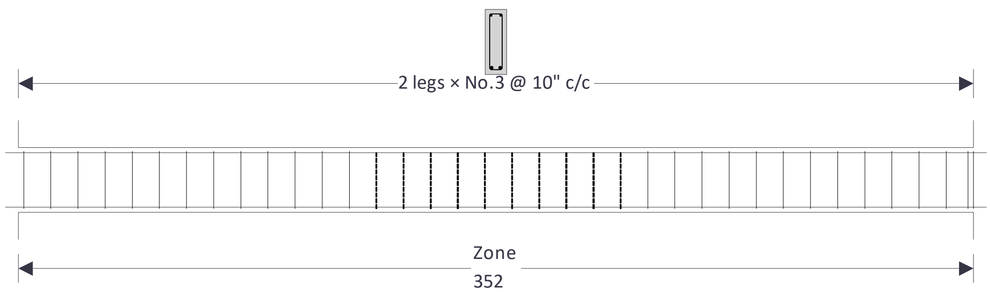
Shear design





Rectangular section in shear

Concrete weight modification factor — λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) — Between 5.45 ft and 23.88 ft

Location where no reinf. is req'd (V_u less ϕV_c / 2) — Between 10.34 ft and 18.99 ft

Maximum reinforcement shear strength — $\phi V_{s,max} = \phi_s \times 8\text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **65.455** kips

Minimum area of shear reinf. (Table 9.6.3.3) — $A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone — d = **21.561** in

Concrete shear strength (eqn. 22.5.5.1) — $\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **16.364** kips

Design shear force within zone — V_u = **21.732** kips

Reinf. shear strength required (eqn. 22.5.1.1) — $\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **5.368** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) — $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.066** in^2/ft

Area of shear reinforcement required — $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided — 2 legs × No.3 @ 10" c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **0.265** in^2/ft

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Maximum longitudinal spacing (Table 9.7.6.2.2) — $s_{vl,max} = d / 2$ = **10.781** in

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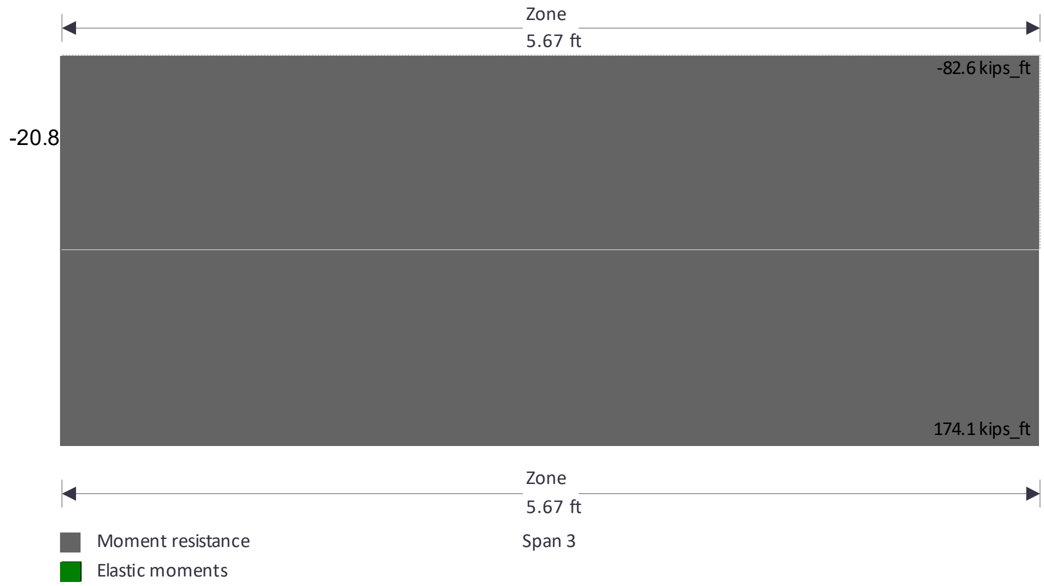
Beam - Span 3

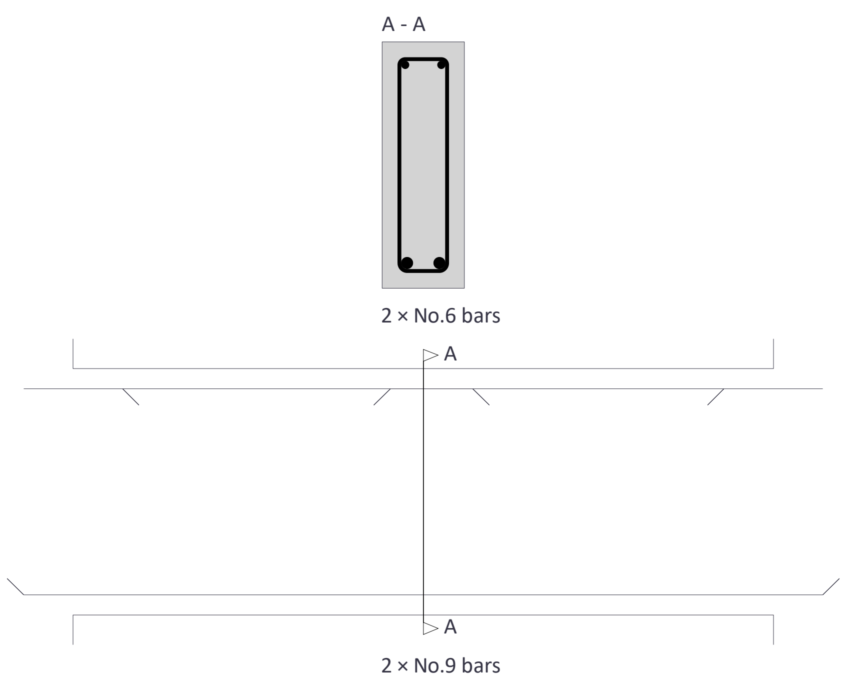
Rectangular section details

Section width b = **8** in

Section depth h = **24** in

Moment design





Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s3\_z2\_min\_red})$ = **20.801** kip_ft

Effective depth to tension reinforcement d = **21.75** in

Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.580** in^2
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in
Depth to neutral axis	$c = a / \beta_1$ = **2.293** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\ in)$ = **0.02546**
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **91.783** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **82.605** kip_ft

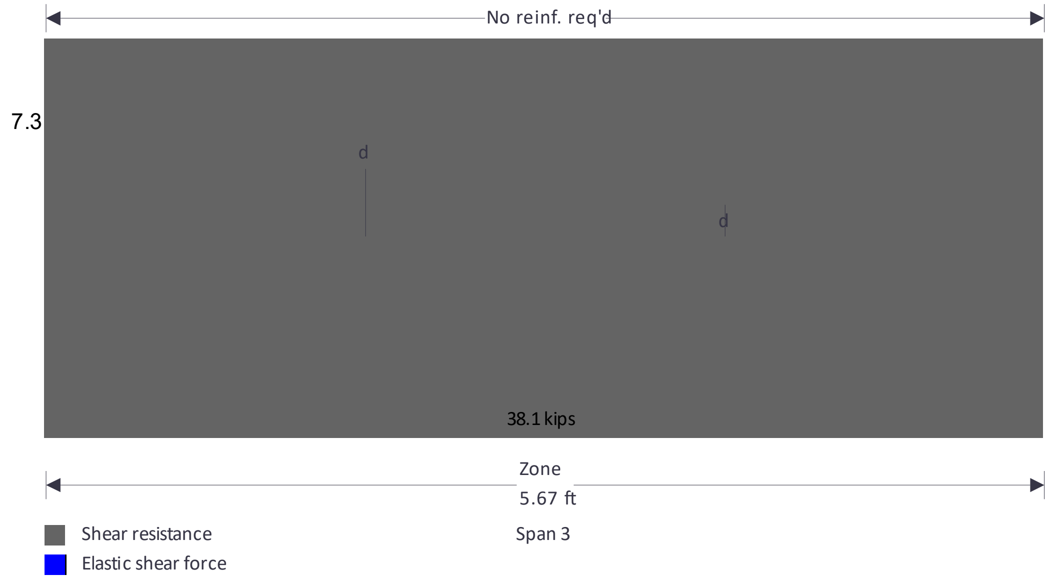
Flexural cracking

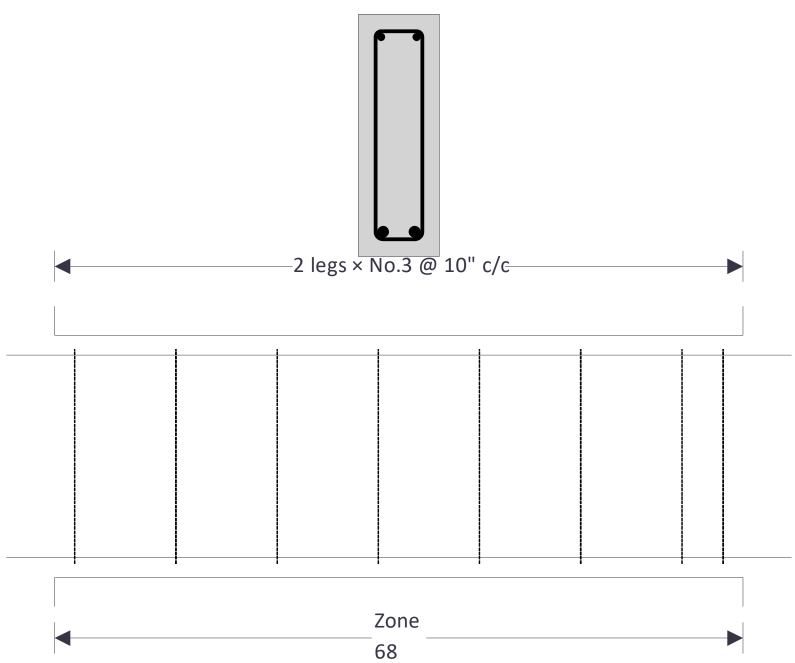
Max. center to center spacing of tension reinf.	$s_{t,max} = s_{top} + \phi_{m1\_s3\_z2\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_t} + \phi_v$ = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	$s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s3\_z2\_v}) + \phi_{m1\_s3\_z2\_t\_L1} \times N_{m1\_s3\_z2\_t\_L1})) / (N_{m1\_s3\_z2\_t\_L1} - 1)$ = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s3\_z2\_v}) + \phi_{m1\_s3\_z2\_b\_L1} \times N_{m1\_s3\_z2\_b\_L1})) / (N_{m1\_s3\_z2\_b\_L1} - 1)$ = **1.994** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	N/A no reinf. required along full length
Location where no reinf. is req'd (V_u less ϕV_c / 2)	Between 0.00 ft and 5.67 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **66.028** kips
Minimum area of shear reinf. (Table 9.6.3.3)	$A_{sv,min}$ = max(50 psi, 0.75 psi $\times \sqrt{(f'_c / 1\text{ psi})}) \times b$ / min(f_y, 60000 psi) = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone	d = **21.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 \text{ psi})} \times b \times d$ = **16.507** kips
Design shear force within zone	V_u = **4.993** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req}$ = $0mm^2/ft$ = **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.3 @ 10" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.265** in^2/ft

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Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas			Job Ref. B190988.00	
	Section Conc Beam Line N Bldg Front			Sheet no./rev. 1	
	Calc. by EJM	Date 11/5/2020	Chk'd by	Date	App'd by	Date

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318

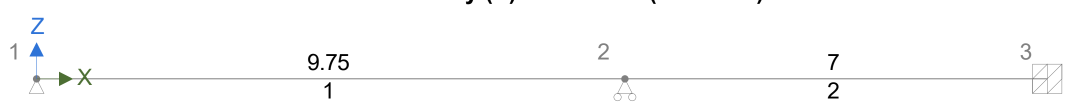
Tedds calculation version 3.3.03

ANALYSIS

Tedds calculation version 1.0.31

Geometry

Geometry (ft) - Concrete (4000 150)

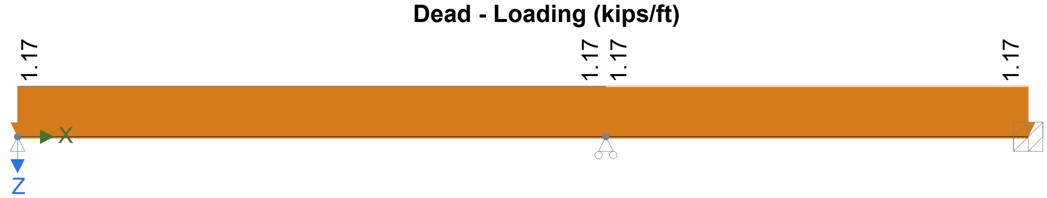


Span	Length (ft)	Section	Start Support	End Support
1	9.75	R 8x16	Pinned	Roller Pin X
2	7	R 8x16	Roller Pin X	Fixed
R 12x20: Area 240 in^2, Inertia Major 8000 in^4, Inertia Minor 2880 in^4, Shear area parallel to Minor 200 in^2, Shear area parallel to Major 200 in^2				
R 8x16: Area 128 in^2, Inertia Major 2731 in^4, Inertia Minor 683 in^4, Shear area parallel to Minor 107 in^2, Shear area parallel to Major 107 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

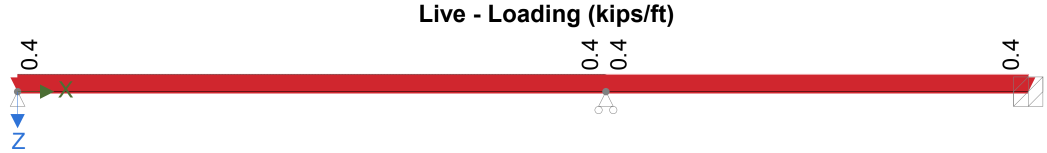
Loading

Self weight included

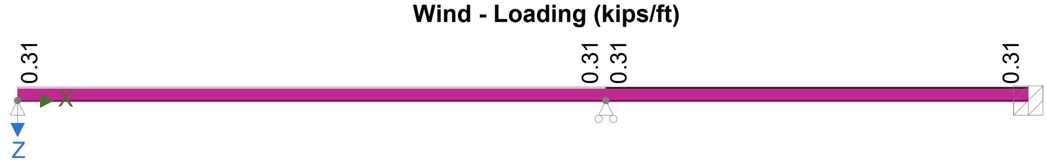
Dead - Loading (kips/ft)



Live - Loading (kips/ft)



Wind - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live	Wind
1.2D + 1.6L (Strength)	1.20	1.20	1.60	
1.2D+1.6L+0.5W (Service)	1.20	1.20	1.60	5.00

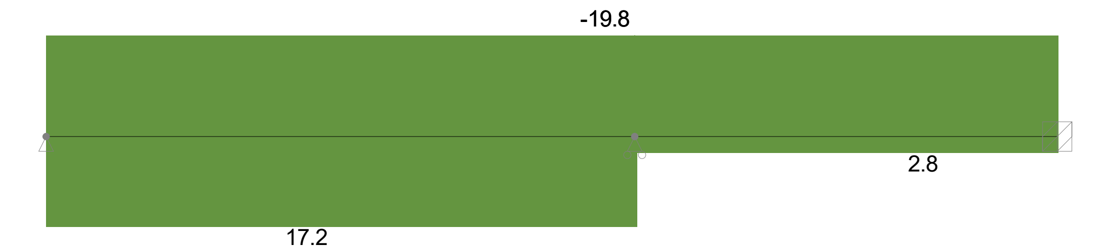
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam on Line N	Dead	UDL	GlobalZ	1.17 kips/ft
Beam on Line N	Live	UDL	GlobalZ	0.4 kips/ft
Beam on Line N	Wind	UDL	GlobalZ	0.31 kips/ft

Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Concrete details

Compressive strength of concrete f'_c = **4000** psi

Density of reinforced concrete w_c = **150** lb / ft^3

Concrete type Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1) $E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi

Strength reduction factor for shear ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Nominal cover to reinforcement

Cover to top reinforcement c_{nom_t} = **1.5** in

Cover to bottom reinforcement c_{nom_b} = **1.5** in

Cover to side reinforcement c_{nom_s} = **1.5** in

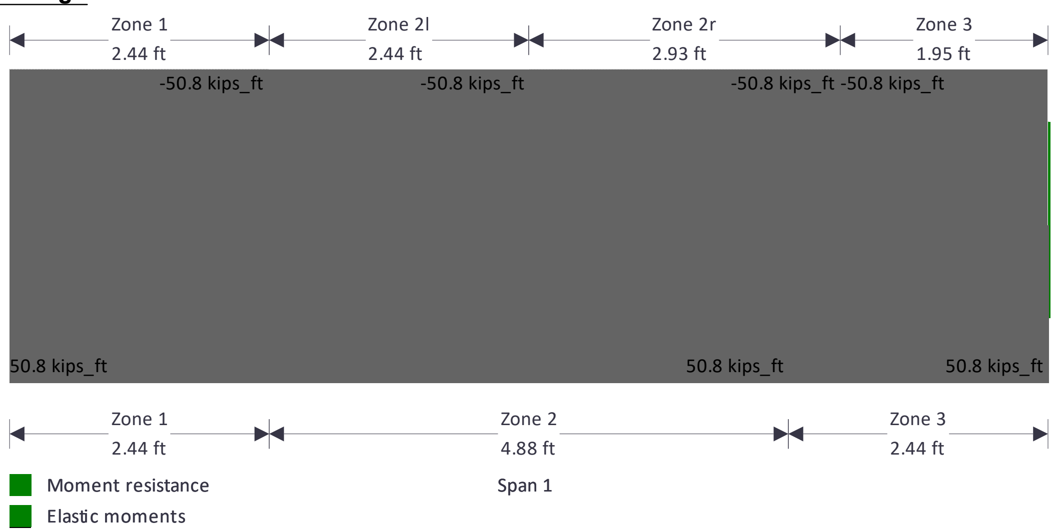
Beam on Line N - Span 1

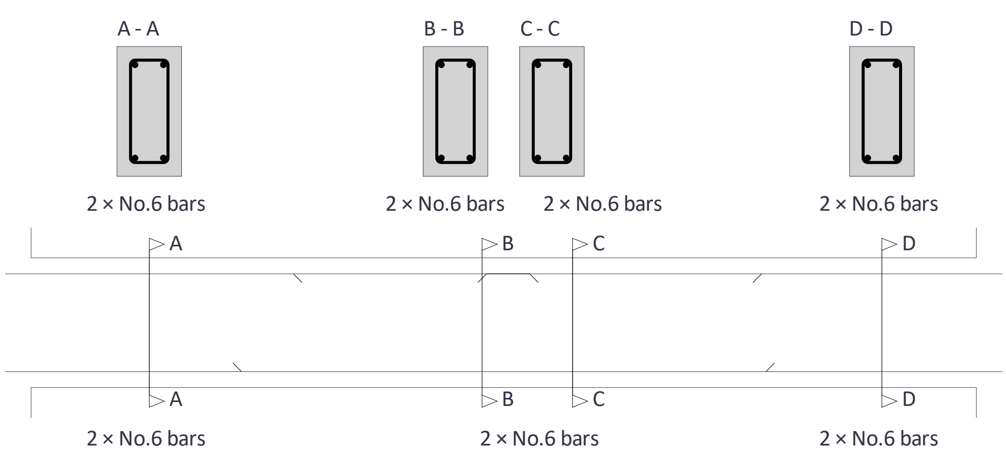
Rectangular section details

Section width b = **8** in

Section depth h = **16** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z1\_max\_red})$ = **25.112** kip_ft

Effective depth to tension reinforcement d = **13.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **1.949** in

Depth to neutral axis c = a / β_1 = **2.293** in

Net tensile strain in extreme tension fibers ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**

Nominal moment strength M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **56.440** kip_ft

Design moment strength ϕM_n = M_n × ϕ_f = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z1\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement c_c = c_{nom_b} + ϕ_v = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

TT T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_t\_L1}$ × $N_{m1\_s1\_z1\_t\_L1}$ + $\phi_{m1\_s1\_z2l\_t\_L1}$ × $N_{m1\_s1\_z2l\_t\_L1}$)) / (($N_{m1\_s1\_z1\_t\_L1}$ + $N_{m1\_s1\_z2l\_t\_L1}$) - 1) = **2.750** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_b\_L1}$ × $N_{m1\_s1\_z1\_b\_L1}$)) / ($N_{m1\_s1\_z1\_b\_L1}$ - 1) = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section M_u = abs($M_{m1\_s1\_z2\_max\_red}$) = **29.450** kip_ft

Effective depth to tension reinforcement d = **13.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **1.949** in

Depth to neutral axis c = a / β_1 = **2.293** in

Net tensile strain in extreme tension fibers ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**

Nominal moment strength M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **56.440** kip_ft

Design moment strength ϕM_n = M_n × ϕ_f = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z2\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v =$ **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **10.313** in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor $K_w =$ **1.00**

Reinforcement yield strength factor $K_f = 0.4 + f_y / 100000$ psi $=$ **1.00**

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f =$ **6.324** in

TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1) =$ **2.750** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} =$ **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} =$ **1.000** in

TT T T T T T

<u>Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)</u>

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_max\_red}) =$ **8.245** kip_ft

Effective depth to tension reinforcement $d =$ **13.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov} =$ **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3 psi \times \sqrt{(f'_c / 1 psi)}, 200 psi) \times b \times d / f_y =$ **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) =$ **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **1.949** in

Depth to neutral axis $c = a / \beta_1 =$ **2.293** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 in) =$ **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **56.440** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f =$ **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1} =$ **3.500** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v =$ **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **10.313** in

TT T T T T T T T

<u>Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)</u>

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_min\_red}) =$ **33.733** kip_ft

Effective depth to tension reinforcement $d =$ **13.75** in

Tension reinforcement provided — 2 × No.6 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) — β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block — a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **1.949** in

Depth to neutral axis — c = a / β_1 = **2.293** in

Net tensile strain in extreme tension fibers — ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) — ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**

Nominal moment strength — M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **56.440** kip_ft

Design moment strength — ϕM_n = M_n × ϕ_f = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{t,max}$ = s_{top} + $\phi_{m1\_s1\_z3\_t\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) — f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement — c_c = c_{nom_t} + ϕ_v = **1.875** in

Maximum allowable top bar spacing (Table 24.3.2) — s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

TT T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing — s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_t\_L1}$ × $N_{m1\_s1\_z3\_t\_L1}$ + $\phi_{m1\_s1\_z2r\_t\_L1}$ × $N_{m1\_s1\_z2r\_t\_L1}$)) / (($N_{m1\_s1\_z3\_t\_L1}$ + $N_{m1\_s1\_z2r\_t\_L1}$) - 1) = **2.750** in

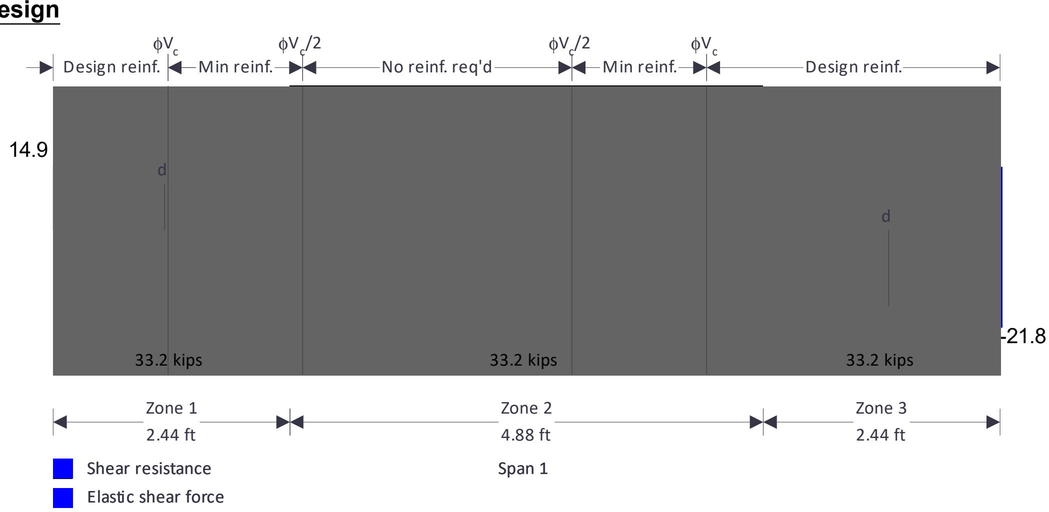
Min. allowable top bar clear spacing (cl.25.2.1) — $s_{top,min}$ = **1.000** in

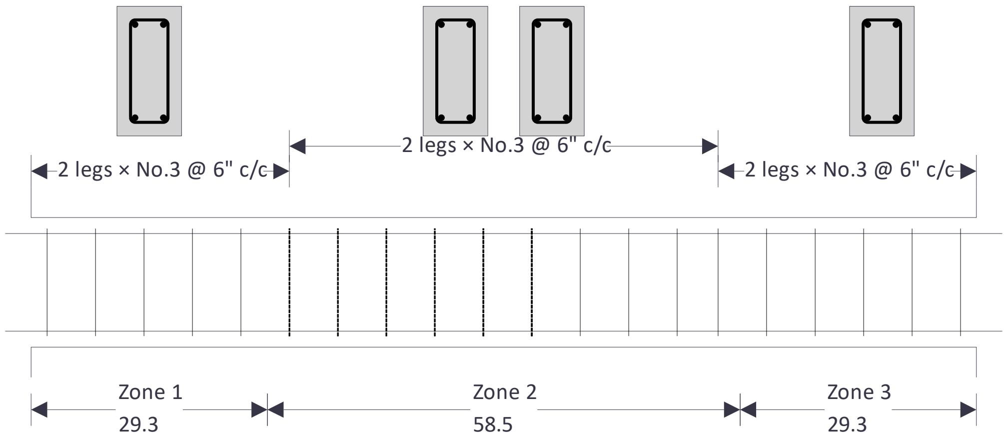
Bottom bar clear spacing — s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_b\_L1}$ × $N_{m1\_s1\_z3\_b\_L1}$)) / ($N_{m1\_s1\_z3\_b\_L1}$ - 1) = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) — $s_{bot,min}$ = **1.000** in

TT T T T T T

Shear design





Rectangular section in shear

Concrete weight modification factor λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) Between 1.18 ft and 6.73 ft

Location where no reinf. is req'd (V_u less ϕV_c / 2) Between 2.57 ft and 5.34 ft

Maximum reinforcement shear strength $\phi V_{s,max} = \phi_s \times 8\text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **41.742** kips

Minimum area of shear reinf. (Table 9.6.3.3) $A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone d = **13.75** in

Concrete shear strength (eqn. 22.5.5.1) $\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **10.436** kips

Design shear force at 13.8in from support V_u = **10.577** kips

Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.141** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.003** in^2/ft

Area of shear reinforcement required $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = d / 2$ = **6.875** in

TT T T T T T T T T T

Zone 2

Effective depth of long. reinf. used for shear zone d = **13.75** in

Concrete shear strength (eqn. 22.5.5.1) $\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **10.436** kips

Design shear force within zone V_u = **12.635** kips

Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **2.199** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.043** in^2/ft

Area of shear reinforcement required $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = d / 2$ = **6.875** in

TT T T T T T T T T T

Zone 3

Effective depth of long. reinf. used for shear zone — d = **13.75** in

Concrete shear strength (eqn. 22.5.5.1) — $\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 \text{ psi})} \times b \times d$ = **10.436** kips

Design shear force at 13.8in from support — V_u = **17.496** kips

Reinf. shear strength required (eqn. 22.5.1.1) — $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **7.061** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) — $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.137** in^2/ft

Area of shear reinforcement required — $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided — 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2) — $s_{vl,max} = d / 2$ = **6.875** in

TT T T T T T T T T T

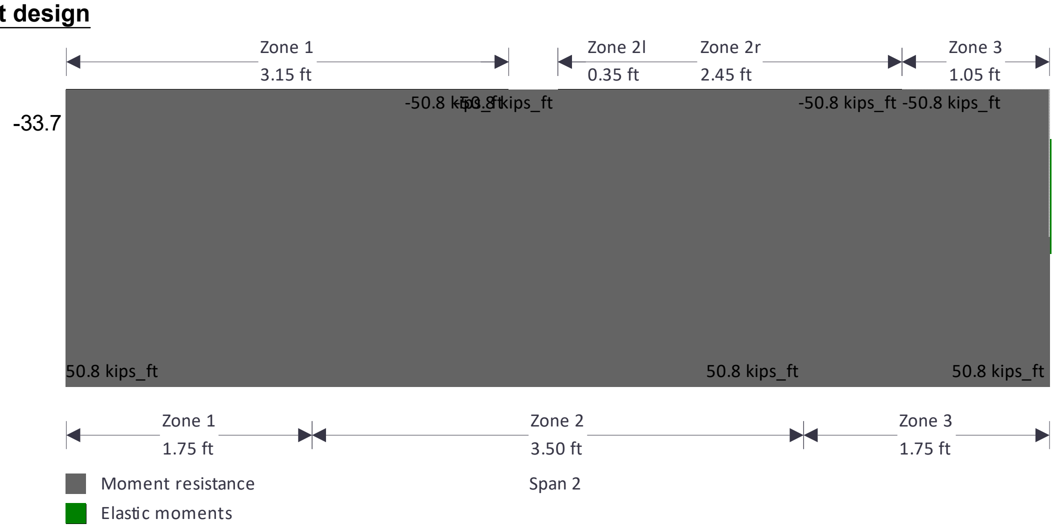
Beam on Line N - Span 2

Rectangular section details

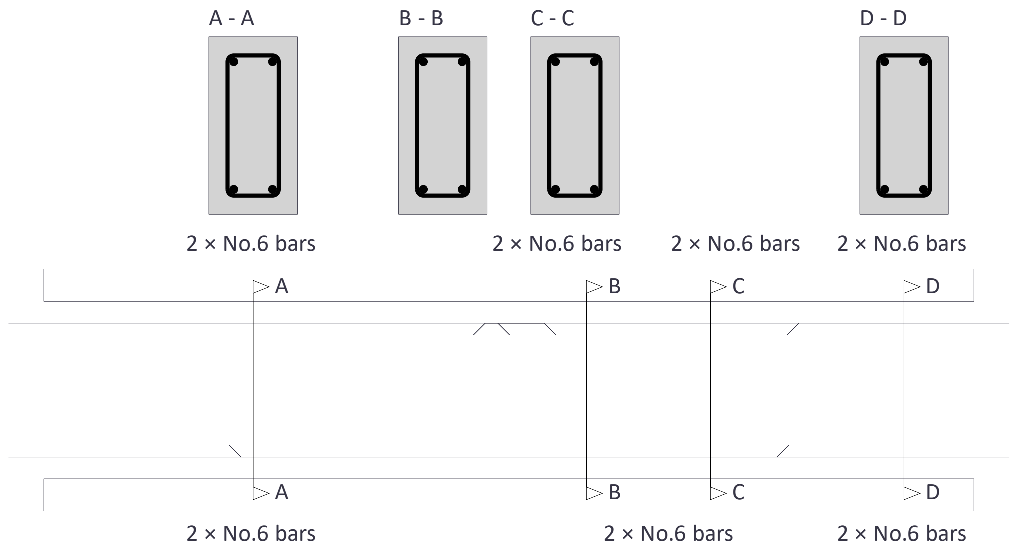
Section width — b = **8** in

Section depth — h = **16** in

Moment design



Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas				Job Ref. B190988.00	
	Section Conc Beam Line N Bldg Front				Sheet no./rev. 9	
	Calc. by EJM	Date 11/5/2020	Chk'd by	Date	App'd by	Date



Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s2\_z1\_min\_red})$ = **33.733** kip_ft
Effective depth to tension reinforcement	d = **13.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.367** in^2
TT T T	T T T T T T T T T
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in
Depth to neutral axis	$c = a / \beta_1$ = **2.293** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.01499**
	T T T T T T
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **56.440** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **50.796** kip_ft
	TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max} = s_{top} + \phi_{m1\_s2\_z1\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_t} + \phi_v$ = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in
TT	T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z1\_v}) + \phi_{m1\_s2\_z1\_t\_L1} \times N_{m1\_s2\_z1\_t\_L1} + \phi_{m1\_s2\_z2l\_t\_L1} \times N_{m1\_s2\_z2l\_t\_L1})) / ((N_{m1\_s2\_z1\_t\_L1} + N_{m1\_s2\_z2l\_t\_L1}) - 1)$ = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z1\_v}) + \phi_{m1\_s2\_z1\_b\_L1} \times N_{m1\_s2\_z1\_b\_L1})) / (N_{m1\_s2\_z1\_b\_L1} - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s2\_z2\_max\_red})$ = **4.735** kip_ft

Effective depth to tension reinforcement d = **13.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in

Depth to neutral axis $c = a / \beta_1$ = **2.293** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) $\phi_f = min(max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **56.440** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f$ = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s2\_z2\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v$ = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor K_w = **1.00**

Reinforcement yield strength factor $K_f = 0.4 + f_y / 100000\ psi$ = **1.00**

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s2} / 18.5) \times K_w \times K_f$ = **4.541** in

TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2r\_t\_L1} \times N_{m1\_s2\_z2r\_t\_L1})) / (N_{m1\_s2\_z2r\_t\_L1} - 1)$ = **2.750** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2\_b\_L1} \times N_{m1\_s2\_z2\_b\_L1} + \phi_{m1\_s2\_z1\_b\_L1} \times N_{m1\_s2\_z1\_b\_L1})) / ((N_{m1\_s2\_z2\_b\_L1} + N_{m1\_s2\_z1\_b\_L1}) - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s2\_z3\_max\_red})$ = **3.735** kip_ft

Effective depth to tension reinforcement d = **13.75** in

Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f$_y$ = **0.367** in^2
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f$_y$ / (0.85 × f'$_c$ × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d$_o$ - c) / max(c, 0.001 in) = **0.01499**
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f$_y$ × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s2\_z3\_b\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f$_s$ = 2/3 × f$_y$ = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **1.875** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f$_s$ - 2.5 × c_c, 12in × 40000psi / f$_s$) = **10.313** in

<u>Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)</u>

Factored bending moment at section	M_u = abs($M_{m1\_s2\_z3\_min\_red}$) = **6.829** kip_ft
Effective depth to tension reinforcement	d = **13.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f$_y$ = **0.367** in^2
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f$_y$ / (0.85 × f'$_c$ × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d$_o$ - c) / max(c, 0.001 in) = **0.01499**
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f$_y$ × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max}$ = s_{top} + $\phi_{m1\_s2\_z3\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f$_s$ = 2/3 × f$_y$ = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_t} + ϕ_v = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f$_s$ - 2.5 × c_c, 12in × 40000psi / f$_s$) = **10.313** in

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z3\_v}) + \phi_{m1\_s2\_z3\_t\_L1} \times N_{m1\_s2\_z3\_t\_L1} + \phi_{m1\_s2\_z2r\_t\_L1} \times N_{m1\_s2\_z2r\_t\_L1})) / ((N_{m1\_s2\_z3\_t\_L1} + N_{m1\_s2\_z2r\_t\_L1}) - 1) =$ **2.750** in

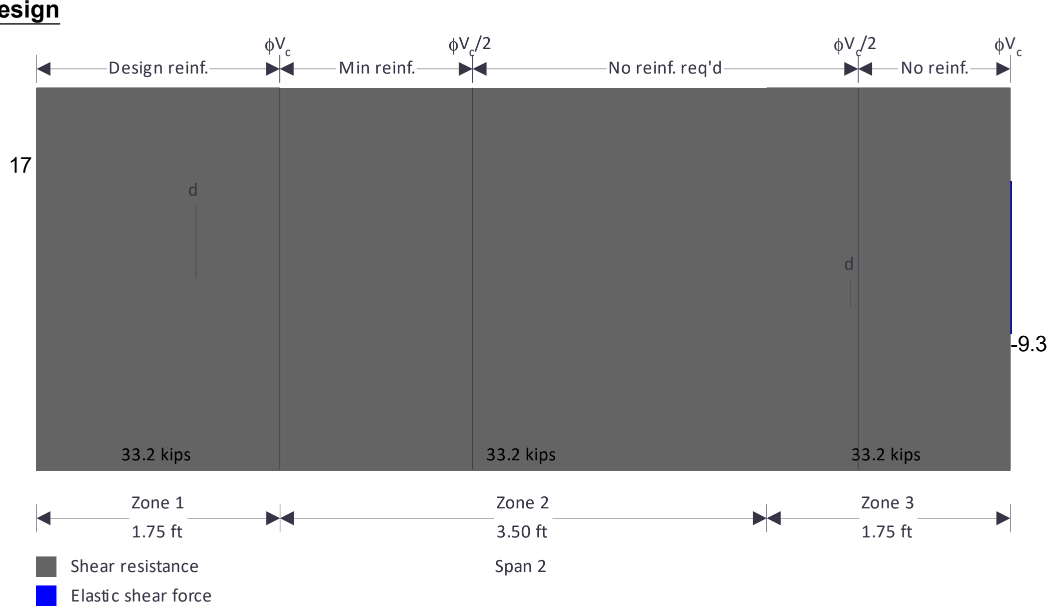
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} =$ **1.000** in

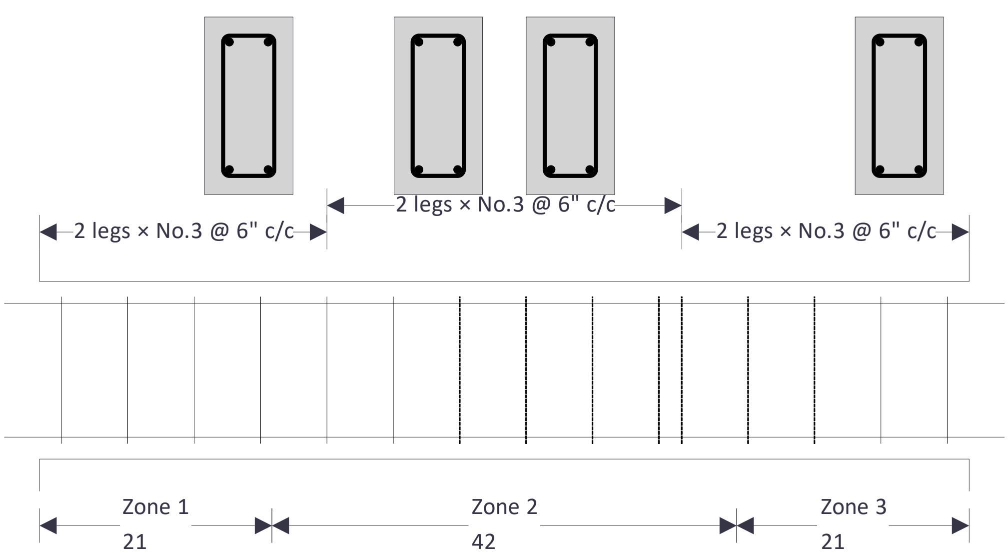
Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z3\_v}) + \phi_{m1\_s2\_z3\_b\_L1} \times N_{m1\_s2\_z3\_b\_L1})) / (N_{m1\_s2\_z3\_b\_L1} - 1) =$ **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} =$ **1.000** in

TT T T T T T

Shear design





Rectangular section in shear

Concrete weight modification factor $\lambda =$ **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c)	Between 1.75 ft and 7.00 ft
Location where no reinf. is req'd (V_u less ϕV_c / 2)	Between 3.13 ft and 7.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8\text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **41.742** kips
Minimum area of shear reinf. (Table 9.6.3.3)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **10.436** kips
Design shear force at 13.8in from support	V_u = **12.704** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **2.269** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.044** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft
Shear reinforcement provided	2 legs × No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max}$ = d / 2 = **6.875** in

Zone 2

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **10.436** kips
Design shear force within zone	V_u = **10.430** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft
Shear reinforcement provided	2 legs × No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max}$ = min(d / 2, 24 in) = **6.875** in

Zone 3

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **10.436** kips
Design shear force at 13.8in from support	V_u = **5.018** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318

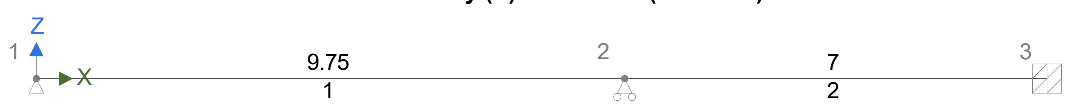
Tedds calculation version 3.3.03

ANALYSIS

Tedds calculation version 1.0.31

Geometry

Geometry (ft) - Concrete (4000 150)

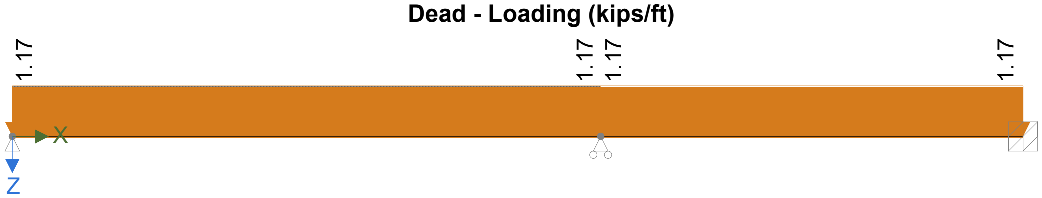


Span	Length (ft)	Section	Start Support	End Support
1	9.75	R 8x16	Pinned	Roller Pin X
2	7	R 8x16	Roller Pin X	Fixed
R 12x20: Area 240 in^2, Inertia Major 8000 in^4, Inertia Minor 2880 in^4, Shear area parallel to Minor 200 in^2, Shear area parallel to Major 200 in^2				
R 8x16: Area 128 in^2, Inertia Major 2731 in^4, Inertia Minor 683 in^4, Shear area parallel to Minor 107 in^2, Shear area parallel to Major 107 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

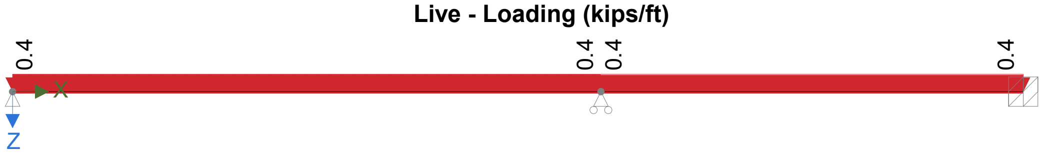
Loading

Self weight included

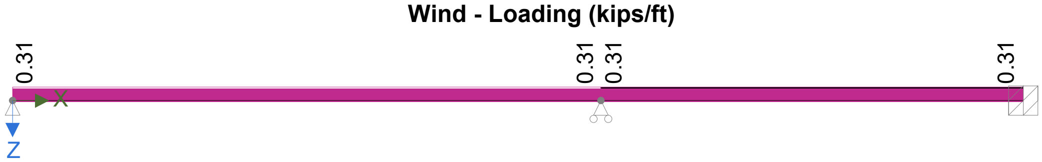
Dead - Loading (kips/ft)



Live - Loading (kips/ft)



Wind - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live	Wind
1.2D + 1.6L (Strength)	1.20	1.20	1.60	
1.2D+1.6L+0.5W (Service)	1.20	1.20	1.60	5.00

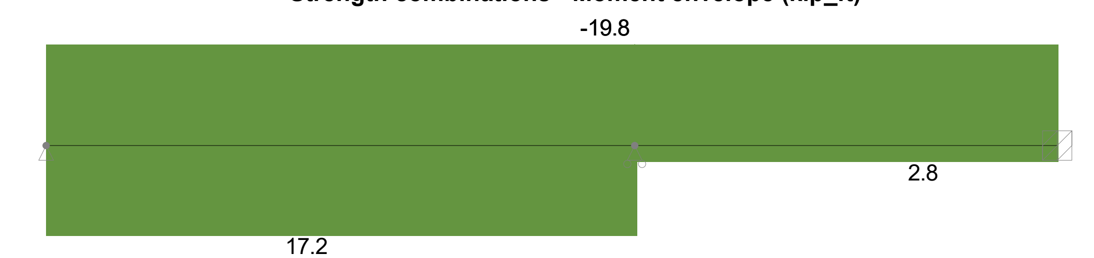
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam on Line N	Dead	UDL	GlobalZ	1.17 kips/ft
Beam on Line N	Live	UDL	GlobalZ	0.4 kips/ft
Beam on Line N	Wind	UDL	GlobalZ	0.31 kips/ft

Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ \text{lb/ft}^3)^{1.5} \times 33\ \text{psi} \times (f'_c / 1\ \text{psi})^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi

Tekla Tedds
GAI Consultants
600 Cranberry Woods Drive
Suite 400
Cranberry Township, PA. 16066

Project				Job Ref.	
I-10 Rest Areas				B190988.00	
Section				Sheet no./rev.	
Conc Beam Line N Bldg Front				3	
Calc. by	Date	Chk'd by	Date	App'd by	Date
EJM	11/5/2020				

Nominal cover to reinforcement

Cover to top reinforcement c_{nom_t} = **1.5** in

Cover to bottom reinforcement c_{nom_b} = **1.5** in

Cover to side reinforcement c_{nom_s} = **1.5** in

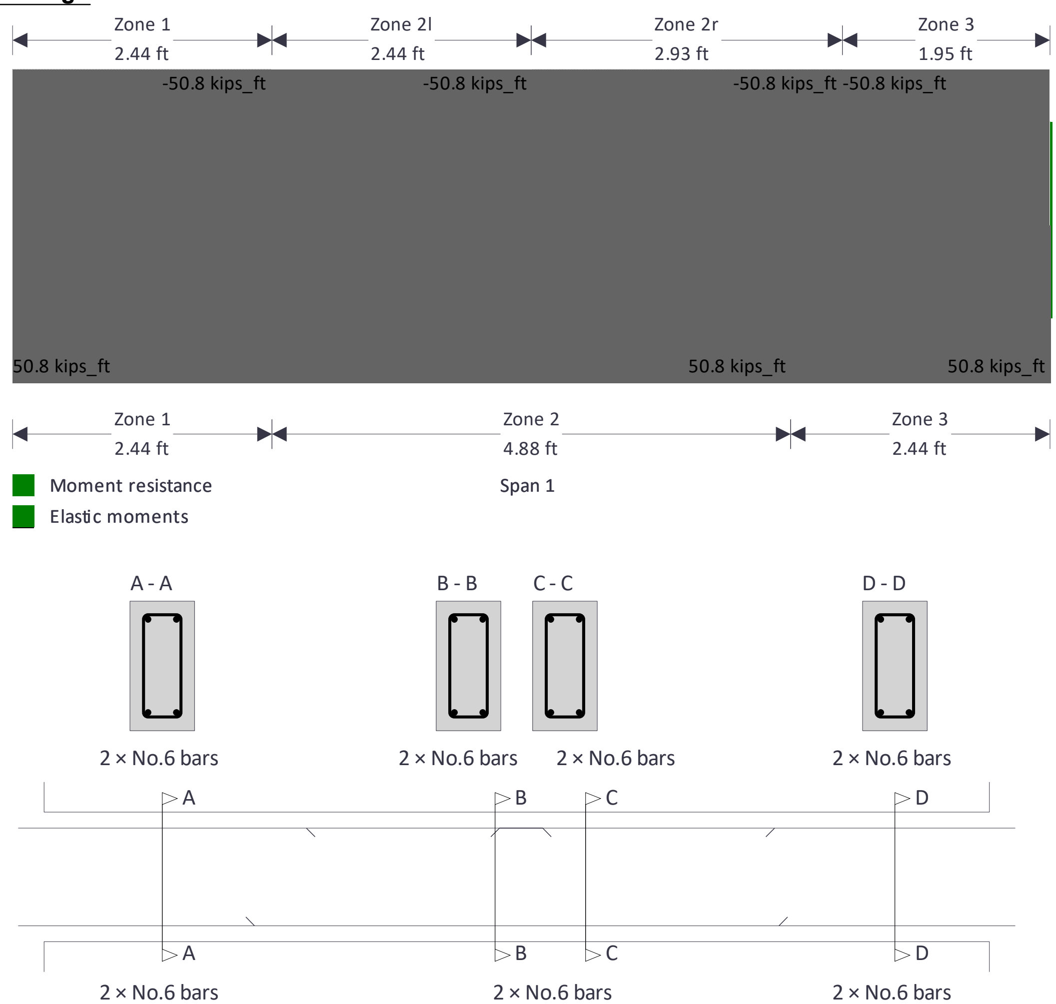
Beam on Line N - Span 1

Rectangular section details

Section width b = **8** in

Section depth h = **16** in

Moment design



Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z1\_max\_red})$ = **25.112** kip_ft

Effective depth to tension reinforcement d = **13.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = max(3 \text{ psi} \times \sqrt{f'_c / 1 \text{ psi}}, 200 \text{ psi}) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'_c × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z1\_b\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **1.875** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

TT T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing	s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_t\_L1}$ × $N_{m1\_s1\_z1\_t\_L1}$ + $\phi_{m1\_s1\_z2l\_t\_L1}$ × $N_{m1\_s1\_z2l\_t\_L1}$)) / (($N_{m1\_s1\_z1\_t\_L1}$ + $N_{m1\_s1\_z2l\_t\_L1}$) - 1) = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_b\_L1}$ × $N_{m1\_s1\_z1\_b\_L1}$)) / ($N_{m1\_s1\_z1\_b\_L1}$ - 1) = **2.750** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s1\_z2\_max\_red}$) = **29.450** kip_ft
Effective depth to tension reinforcement	d = **13.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'_c / 1 psi), 200 psi) × b × d / f_y = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'_c × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z2\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v =$ **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **10.313** in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor $K_w =$ **1.00**

Reinforcement yield strength factor $K_f = 0.4 + f_y / 100000$ psi $=$ **1.00**

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f =$ **6.324** in

TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1) =$ **2.750** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} =$ **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} =$ **1.000** in

TT T T T T T

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_max\_red}) =$ **8.245** kip_ft

Effective depth to tension reinforcement $d =$ **13.75** in

Tension reinforcement provided $2 \times$ No.6 bars

Area of tension reinforcement provided $A_{s,prov} =$ **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}, 200 \text{ psi}) \times b \times d / f_y =$ **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **1.949** in

Depth to neutral axis $c = a / \beta_1 =$ **2.293** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) =$ **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **56.440** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f =$ **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1} =$ **3.500** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v =$ **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **10.313** in

TT T T T T T T T

Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_min\_red}) =$ **33.733** kip_ft

Effective depth to tension reinforcement $d =$ **13.75** in

Tension reinforcement provided: 2 × No.6 bars

Area of tension reinforcement provided: $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2): $A_{s,min} = \max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3): $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block: $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in

Depth to neutral axis: $c = a / \beta_1$ = **2.293** in

Net tensile strain in extreme tension fibers: $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\ in)$ = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1): $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**

Nominal moment strength: $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **56.440** kip_ft

Design moment strength: $\phi M_n = M_n \times \phi_f$ = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.: $s_{t,max} = s_{top} + \phi_{m1\_s1\_z3\_t\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2): $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement: $c_c = c_{nom\_t} + \phi_v$ = **1.875** in

Maximum allowable top bar spacing (Table 24.3.2): $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

TT T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing: $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1)$ = **2.750** in

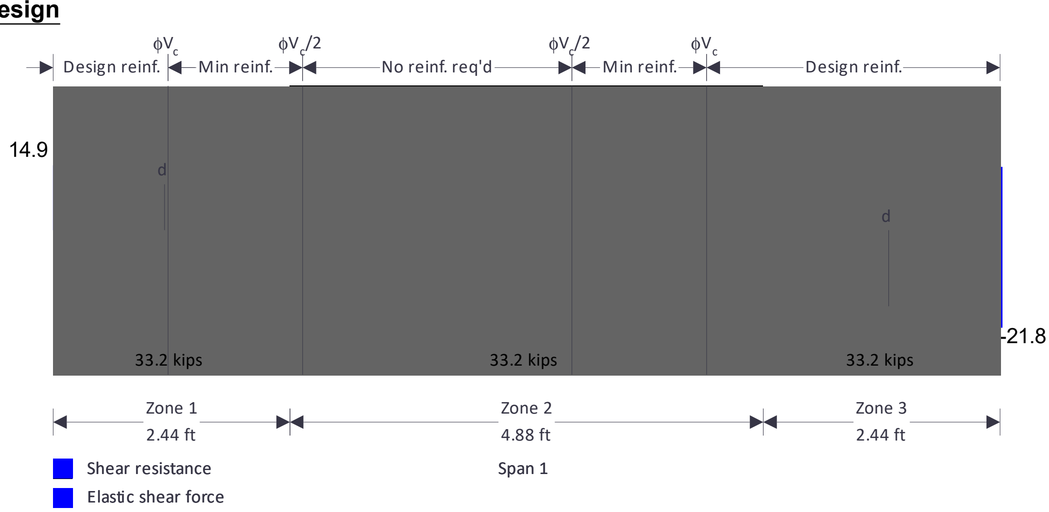
Min. allowable top bar clear spacing (cl.25.2.1): $s_{top,min}$ = **1.000** in

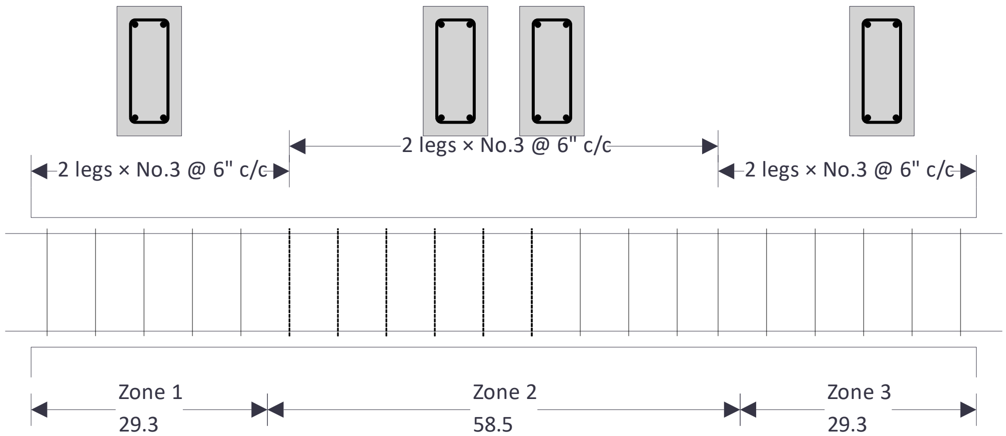
Bottom bar clear spacing: $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1): $s_{bot,min}$ = **1.000** in

TT T T T T T

Shear design





Rectangular section in shear

Concrete weight modification factor — λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) — Between 1.18 ft and 6.73 ft

Location where no reinf. is req'd (V_u less ϕV_c / 2) — Between 2.57 ft and 5.34 ft

Maximum reinforcement shear strength — $\phi V_{s,max} = \phi_s \times 8 \text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1 \text{ psi})} \times b \times d$ = **41.742** kips

Minimum area of shear reinf. (Table 9.6.3.3) — $A_{sv,min} = \max(50 \text{ psi}, 0.75 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}) \times b / \min(f_y, 60000 \text{ psi})$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone — d = **13.75** in

Concrete shear strength (eqn. 22.5.5.1) — $\phi V_c = \phi_s \times \lambda \times 2 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})} \times b \times d$ = **10.436** kips

Design shear force at 13.8in from support — V_u = **10.577** kips

Reinf. shear strength required (eqn. 22.5.1.1) — $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **0.141** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) — $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.003** in^2/ft

Area of shear reinforcement required — $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided — 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2) — $s_{vl,max}$ = d / 2 = **6.875** in

TT T T T T T T T T T

Zone 2

Effective depth of long. reinf. used for shear zone — d = **13.75** in

Concrete shear strength (eqn. 22.5.5.1) — $\phi V_c = \phi_s \times \lambda \times 2 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})} \times b \times d$ = **10.436** kips

Design shear force within zone — V_u = **12.635** kips

Reinf. shear strength required (eqn. 22.5.1.1) — $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **2.199** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) — $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.043** in^2/ft

Area of shear reinforcement required — $A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft

Shear reinforcement provided — 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2) — $s_{vl,max}$ = d / 2 = **6.875** in

TT T T T T T T T T T

Zone 3

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 \text{ psi})} \times b \times d$ = **10.436** kips
Design shear force at 13.8in from support	V_u = **17.496** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **7.061** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.137** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft
Shear reinforcement provided	2 legs × No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2$ = **6.875** in

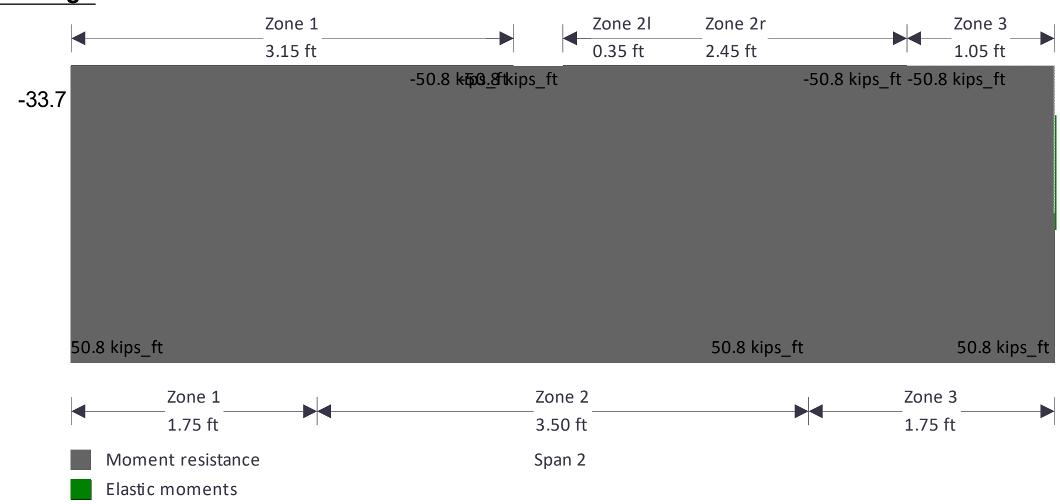
TT T T T T T T T T T

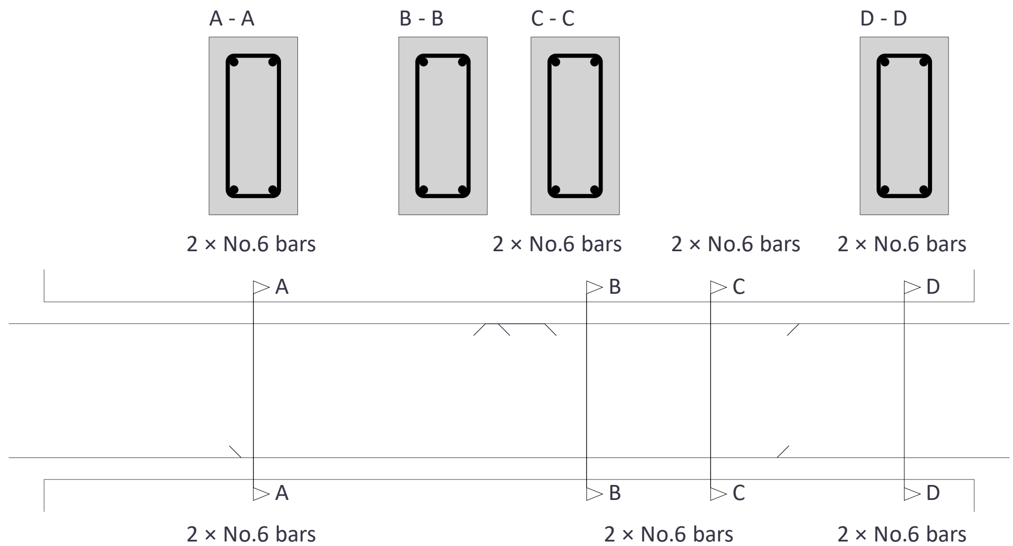
Beam on Line N - Span 2

Rectangular section details

Section width	b = **8** in
Section depth	h = **16** in

Moment design





Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s2\_z1\_min\_red})$ = **33.733** kip_ft
Effective depth to tension reinforcement	d = **13.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3 psi \times \sqrt{(f'_c / 1 psi)}, 200 psi) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in
Depth to neutral axis	$c = a / \beta_1$ = **2.293** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001 in)$ = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **56.440** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max} = s_{top} + \phi_{m1\_s2\_z1\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_t} + \phi_v$ = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

TT T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z1\_v}) + \phi_{m1\_s2\_z1\_t\_L1} \times N_{m1\_s2\_z1\_t\_L1} + \phi_{m1\_s2\_z2l\_t\_L1} \times N_{m1\_s2\_z2l\_t\_L1})) / ((N_{m1\_s2\_z1\_t\_L1} + N_{m1\_s2\_z2l\_t\_L1}) - 1)$ = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in

Bottom bar clear spacing — $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z1\_v}) + \phi_{m1\_s2\_z1\_b\_L1} \times N_{m1\_s2\_z1\_b\_L1})) / (N_{m1\_s2\_z1\_b\_L1} - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) — $s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — $M_u = abs(M_{m1\_s2\_z2\_max\_red})$ = **4.735** kip_ft

Effective depth to tension reinforcement — d = **13.75** in

Tension reinforcement provided — 2 × No.6 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min} = \max(3 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}, 200 \text{ psi}) \times b \times d / f_y$ = **0.367** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block — $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in

Depth to neutral axis — $c = a / \beta_1$ = **2.293** in

Net tensile strain in extreme tension fibers — $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in})$ = **0.01499**

T T T T T T

Strength reduction factor (cl.21.2.1) — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**

Nominal moment strength — $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **56.440** kip_ft

Design moment strength — $\phi M_n = M_n \times \phi_f$ = **50.796** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{b,max} = s_{bot} + \phi_{m1\_s2\_z2\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) — $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement — $c_c = c_{nom\_b} + \phi_v$ = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) — $s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s)$ = **10.313** in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor — K_w = **1.00**

Reinforcement yield strength factor — $K_f = 0.4 + f_y / 100000 \text{ psi}$ = **1.00**

Minimum thickness of beam (Table 9.3.1.1) — $h_{min} = (L_{m1\_s2} / 18.5) \times K_w \times K_f$ = **4.541** in

TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing — $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2r\_t\_L1} \times N_{m1\_s2\_z2r\_t\_L1})) / (N_{m1\_s2\_z2r\_t\_L1} - 1)$ = **2.750** in

Min. allowable top bar clear spacing (cl.25.2.1) — $s_{top,min}$ = **1.000** in

Bottom bar clear spacing — $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2\_b\_L1} \times N_{m1\_s2\_z2\_b\_L1} + \phi_{m1\_s2\_z1\_b\_L1} \times N_{m1\_s2\_z1\_b\_L1})) / ((N_{m1\_s2\_z2\_b\_L1} + N_{m1\_s2\_z1\_b\_L1}) - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) — $s_{bot,min}$ = **1.000** in

TT T T T T T

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — $M_u = abs(M_{m1\_s2\_z3\_max\_red})$ = **3.735** kip_ft

Effective depth to tension reinforcement — d = **13.75** in

Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f$_y$ = **0.367** in^2
TT T T	T T T T T T T T T
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f$_y$ / (0.85 × f'$_c$ × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d$_o$ - c) / max(c, 0.001 in) = **0.01499**
	T T T T T T
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f$_y$ × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft
	TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s2\_z3\_b\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f$_s$ = 2/3 × f$_y$ = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **1.875** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f$_s$ - 2.5 × c$_c$, 12in × 40000psi / f$_s$) = **10.313** in
TT	T T T T T T T

<u>**Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)**</u>

Factored bending moment at section	M_u = abs($M_{m1\_s2\_z3\_min\_red}$) = **6.829** kip_ft
Effective depth to tension reinforcement	d = **13.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f$_y$ = **0.367** in^2
TT T T	T T T T T T T T T
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f$_y$ / (0.85 × f'$_c$ × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d$_o$ - c) / max(c, 0.001 in) = **0.01499**
	T T T T T T
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f$_y$ × (d - a / 2) = **56.440** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **50.796** kip_ft
	TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max}$ = s_{top} + $\phi_{m1\_s2\_z3\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f$_s$ = 2/3 × f$_y$ = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_t} + ϕ_v = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f$_s$ - 2.5 × c$_c$, 12in × 40000psi / f$_s$) = **10.313** in
TT	T T T T T T T

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z3\_v}) + \phi_{m1\_s2\_z3\_t\_L1} \times N_{m1\_s2\_z3\_t\_L1} + \phi_{m1\_s2\_z2r\_t\_L1} \times N_{m1\_s2\_z2r\_t\_L1})) / ((N_{m1\_s2\_z3\_t\_L1} + N_{m1\_s2\_z2r\_t\_L1}) - 1)$ = **2.750** in

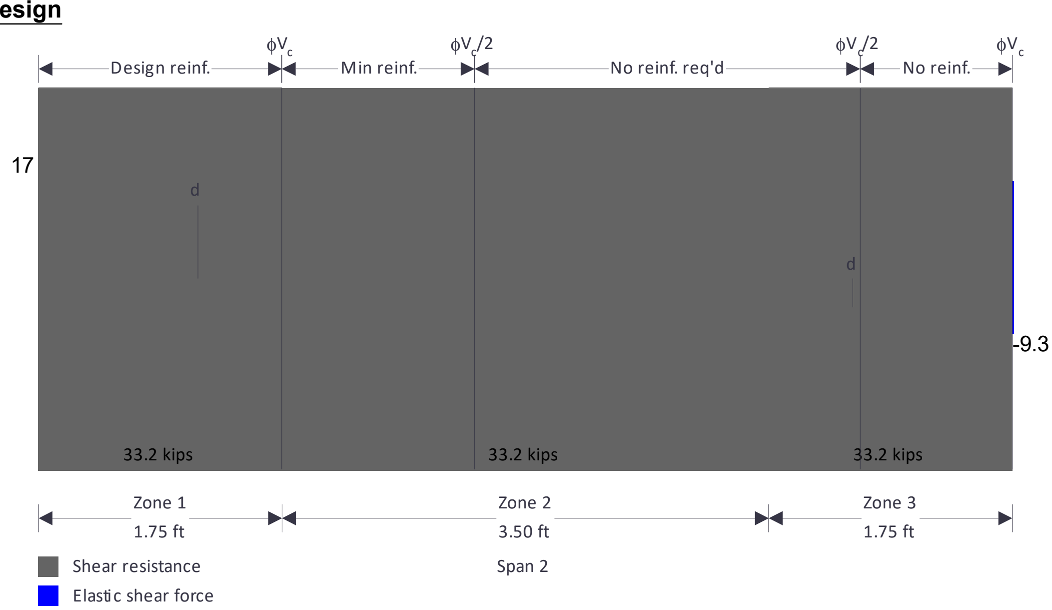
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

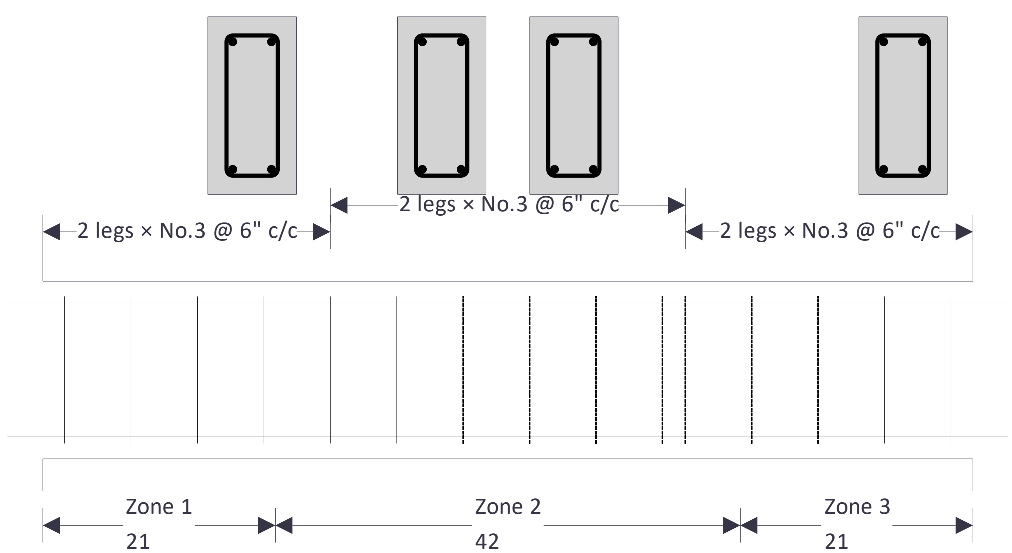
Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z3\_v}) + \phi_{m1\_s2\_z3\_b\_L1} \times N_{m1\_s2\_z3\_b\_L1})) / (N_{m1\_s2\_z3\_b\_L1} - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

TT T T T T T

Shear design





Rectangular section in shear

Concrete weight modification factor λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c)	Between 1.75 ft and 7.00 ft
Location where no reinf. is req'd (V_u less ϕV_c / 2)	Between 3.13 ft and 7.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000psi) / 1 psi)} \times b \times d$ = **41.742** kips
Minimum area of shear reinf. (Table 9.6.3.3)	$A_{sv,min}$ = max(50 psi, 0.75 psi $\times \sqrt{(f'_c / 1 psi)}) \times b$ / min(f_y, 60000 psi) = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 psi)} \times b \times d$ = **10.436** kips
Design shear force at 13.8in from support	V_u = **12.704** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **2.269** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 psi) \times d)$ = **0.044** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max}$ = d / 2 = **6.875** in

TT T T T T T T T T T

Zone 2

Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 psi)} \times b \times d$ = **10.436** kips
Design shear force within zone	V_u = **10.430** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 psi) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.080** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

TT T T T T T T T T T T

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max}$ = min(d / 2, 24 in) = **6.875** in

TT T T T T T T T T T

Zone 3

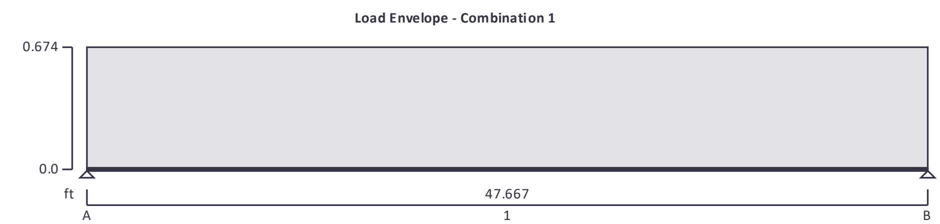
Effective depth of long. reinf. used for shear zone	d = **13.75** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(f'_c / 1 psi)} \times b \times d$ = **10.436** kips
Design shear force at 13.8in from support	V_u = **5.018** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 psi) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

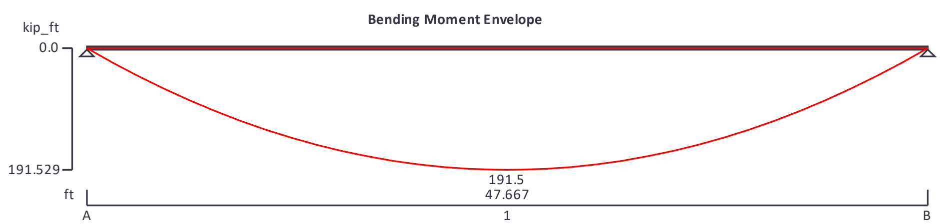
TT T T T Tϕ T T

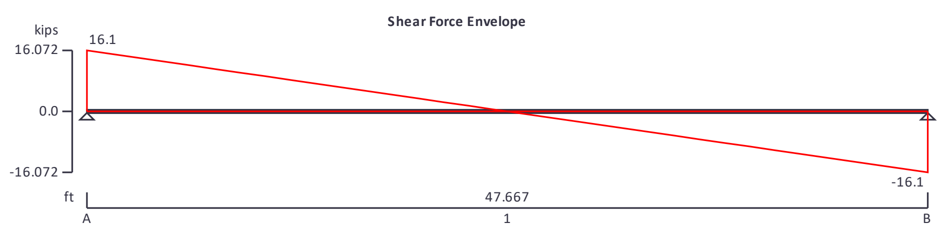
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead - Dead full UDL 0.261 kips/ft
	Live - Roof Live full UDL 0.157 kips/ft
	Wind - Wind full UDL 0.056 kips/ft

Load combinations

Load combination 1 - DL+LL	Support A	Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50

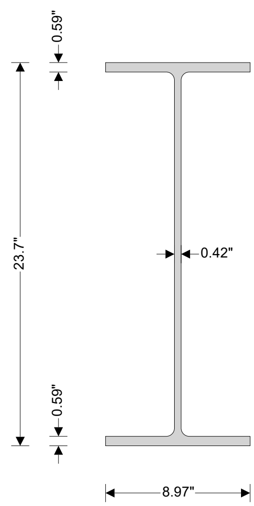
		Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50
	Support B	Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50

Analysis results

Maximum moment	M_{max} = **191.5** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **16.1** kips	V_{min} = **-16.1** kips
Deflection	δ_{max} = **1.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **16.1** kips	R_{A_min} = **16.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **7.8** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ Live}$ = **3.7** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **1.3** kips	
Maximum reaction at support B	R_{B_max} = **16.1** kips	R_{B_min} = **16.1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **7.8** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ Live}$ = **3.7** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **1.3** kips	

Section details

Section type	**W 24x68 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f) = \mathbf{7.67}$

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]} = \mathbf{9.15}$

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]} = \mathbf{24.08}$ Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w = \mathbf{51.86}$

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = \mathbf{90.55}$

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} = \mathbf{137.27}$ Compact

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{16.072}$ kips

Web area $A_w = d \times t_w = \mathbf{9.835}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5.34}$

Web shear coefficient - eq G2-3 $C_{v1} = \mathbf{1}$

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1} = \mathbf{295.065}$ kips

Resistance factor for shear $\phi_v = \mathbf{1.00}$

Design shear strength $V_c = \phi_v \times V_n = \mathbf{295.065}$ kips

Design of members for flexure in the major axis - Chapter F

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{191.529}$ kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x = \mathbf{737.5}$ kips_ft

Nominal flexural strength $M_n = M_{nyld} = \mathbf{737.500}$ kips_ft

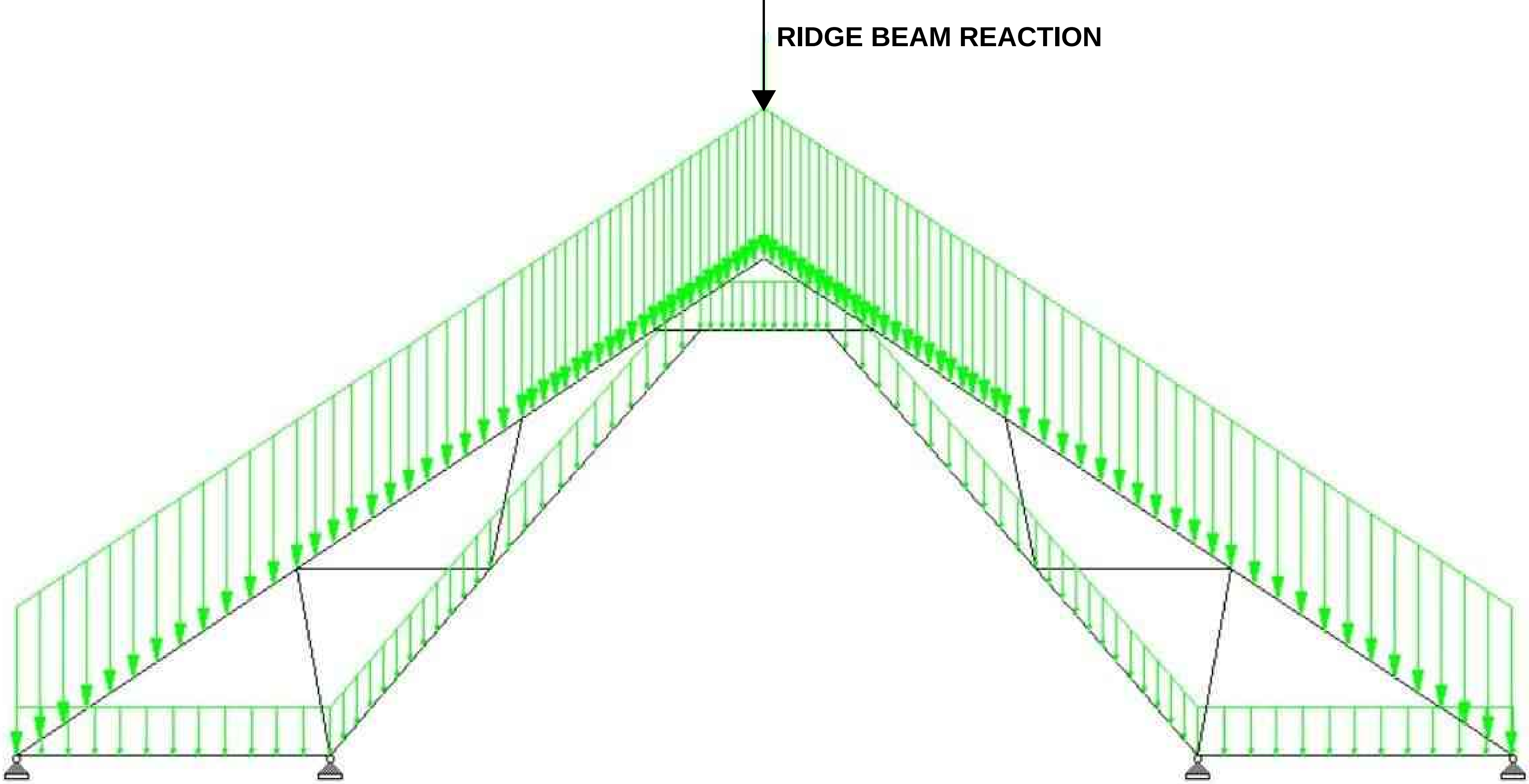
Design flexural strength $M_c = \phi_b \times M_n = \mathbf{663.750}$ kips_ft

Design of members for vertical deflection

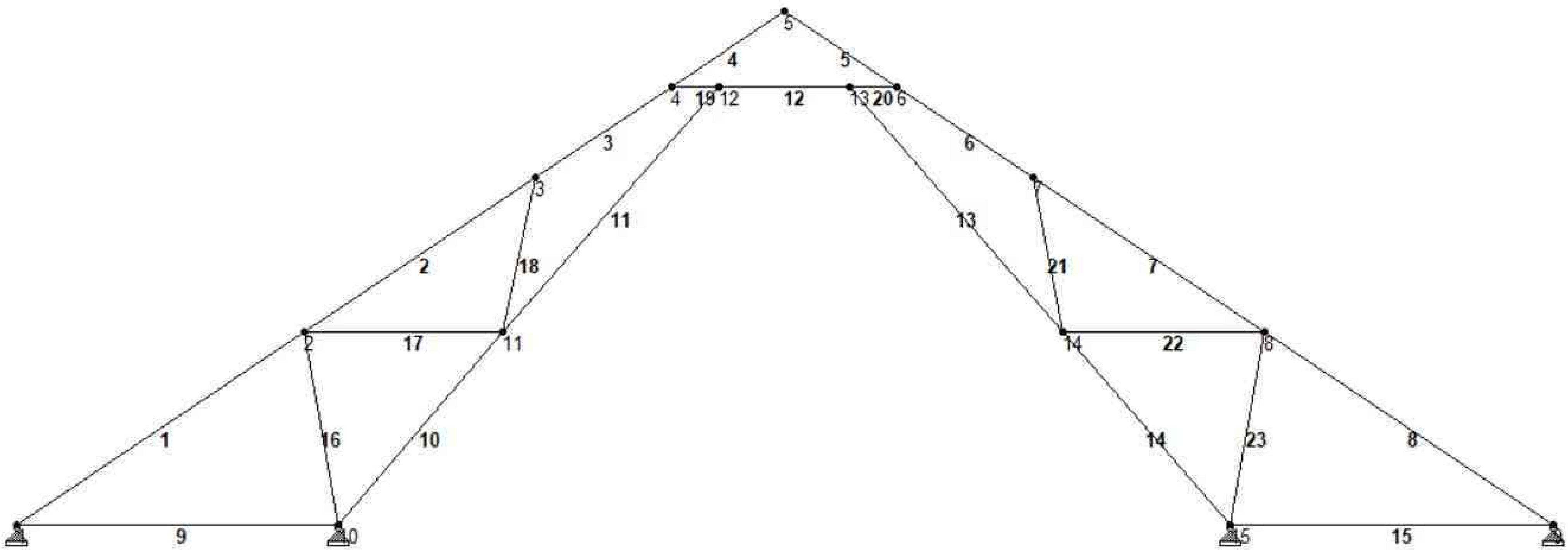
Consider deflection due to dead, roof live and wind loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 = \mathbf{2.288}$ in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{1.186}$ in



TUBE TRUSS LOADING (DL)



TUBE TRUSS JOINTS AND MEMBERS

STAAD.PRO CODE CHECKING - (AISC-360-10-ASD) v1.4a
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE Noted)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Sec. E1	0.055	103
		6.58 C	0.00	0.38	0.00
2 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Sec. E1	0.057	100
		7.79 C	0.00	3.69	0.00
3 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.110	103
		9.11 C	0.00	-26.89	52.64
4 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.131	100
		17.59 C	0.00	25.06	0.00
5 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.132	100
		17.59 C	0.00	25.32	43.73
6 ST	HSST6X4X0.375		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.108	103
		9.09 C	0.00	-26.12	0.00

```
  7 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Sec. E1            0.058       100
                  7.88 C         0.00           3.78      89.75

  8 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Sec. E1            0.055       103
                  6.66 C         0.00          -0.02     111.50

  9 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b          0.010       100
                  0.00 T         0.00          -3.44      34.67

 10 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Sec. E1            0.073       100
                 10.31 C         0.00           3.04       0.00

 11 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b          0.063       103
                  6.57 C         0.00         -11.97     105.16

 12 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b          0.049       103
                  2.67 T         0.00          13.57      42.20

 13 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b          0.062       103
                  6.63 C         0.00         -11.52       0.00
```

```
 14 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Sec. E1             0.073       100
                 10.32 C         0.00           3.13       81.98

 15 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b           0.011       104
                  0.00 T         0.00          -3.70       69.34

 16 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Sec. E1             0.012       100
                  1.79 C         0.00           0.24        0.00

 17 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b           0.019       100
                  1.38 T         0.00           4.83        0.00

 18 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b           0.053       103
                  2.30 C         0.00          15.14       50.85

 19 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b           0.177       103
                  6.92 T         0.00         -51.37        0.00

 20 ST   HSST6X4X0.375            (AISC SECTIONS)
                     PASS   Eq. H1-1b           0.176       103
                  6.94 T         0.00         -51.07       15.28
```

```
  21 ST   HSST6X4X0.375           (AISC SECTIONS)
                      PASS   Eq. H1-1b           0.052       103
                   2.14 C          0.00         14.93        0.00

  22 ST   HSST6X4X0.375           (AISC SECTIONS)
                      PASS   Eq. H1-1b           0.019       104
                   1.56 T          0.00          4.83       65.04

  23 ST   HSST6X4X0.375           (AISC SECTIONS)
                      PASS   Sec. E1             0.013       104
                   2.02 C          0.00          0.03       62.86
```

130. UNIT FEET POUND
131. STEEL TAKE OFF ALL

Job Information

	Engineer	Checked	Approved
Name:	EJM	NJA	
Date:	6-June-2020	6-15-2020	

Project ID	
Project Name	

Structure Type	PLANE FRAME

Number of Nodes	15	Highest Node	15
Number of Elements	23	Highest Beam	23

Number of Basic Load Cases	-2
Number of Combination Load Cases	0

All	The Whole Structure

Type	L/C	Name
Primary	1	DL
Primary	2	DL EXTERNAL
Primary	3	LL
Primary	4	WL NEG
Primary	5	WL B
Primary	100	DL+LL
Primary	101	DL+0.6WL (A)
Primary	102	DL+0.6WL (B)
Primary	103	DL+0.75LL+0.45WL (A)
Primary	104	DL+0.75LL+0.45WL (B)
Primary	105	0.6DL+0.6WL (A)
Primary	106	0.6DL+0.6WL (B)

Utilization Ratio

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
1	HSST6X4X0	HSST6X4X0	0.055	1.000	0.055	Sec. E1	103	6.180	28.300	14.900	32.800
2	HSST6X4X0	HSST6X4X0	0.057	1.000	0.057	Sec. E1	100	6.180	28.300	14.900	32.800
3	HSST6X4X0	HSST6X4X0	0.110	1.000	0.110	Eq. H1-1b	103	6.180	28.300	14.900	32.800
4	HSST6X4X0	HSST6X4X0	0.131	1.000	0.131	Eq. H1-1b	100	6.180	28.300	14.900	32.800
5	HSST6X4X0	HSST6X4X0	0.132	1.000	0.132	Eq. H1-1b	100	6.180	28.300	14.900	32.800
6	HSST6X4X0	HSST6X4X0	0.108	1.000	0.108	Eq. H1-1b	103	6.180	28.300	14.900	32.800
7	HSST6X4X0	HSST6X4X0	0.058	1.000	0.058	Sec. E1	100	6.180	28.300	14.900	32.800
8	HSST6X4X0	HSST6X4X0	0.055	1.000	0.055	Sec. E1	103	6.180	28.300	14.900	32.800

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
12	HSST6X4X0	HSST6X4X0	0.049	1.000	0.049	Eq. H1-1b	103	6.180	28.300	14.900	32.800
13	HSST6X4X0	HSST6X4X0	0.062	1.000	0.062	Eq. H1-1b	103	6.180	28.300	14.900	32.800
14	HSST6X4X0	HSST6X4X0	0.073	1.000	0.073	Sec. E1	100	6.180	28.300	14.900	32.800
15	HSST6X4X0	HSST6X4X0	0.011	1.000	0.011	Eq. H1-1b	104	6.180	28.300	14.900	32.800
16	HSST6X4X0	HSST6X4X0	0.012	1.000	0.012	Sec. E1	100	6.180	28.300	14.900	32.800
17	HSST6X4X0	HSST6X4X0	0.019	1.000	0.019	Eq. H1-1b	100	6.180	28.300	14.900	32.800
18	HSST6X4X0	HSST6X4X0	0.053	1.000	0.053	Eq. H1-1b	103	6.180	28.300	14.900	32.800
19	HSST6X4X0	HSST6X4X0	0.177	1.000	0.177	Eq. H1-1b	103	6.180	28.300	14.900	32.800
20	HSST6X4X0	HSST6X4X0	0.176	1.000	0.176	Eq. H1-1b	103	6.180	28.300	14.900	32.800
21	HSST6X4X0	HSST6X4X0	0.052	1.000	0.052	Eq. H1-1b	103	6.180	28.300	14.900	32.800
22	HSST6X4X0	HSST6X4X0	0.019	1.000	0.019	Eq. H1-1b	104	6.180	28.300	14.900	32.800
23	HSST6X4X0	HSST6X4X0	0.013	1.000	0.013	Sec. E1	104	6.180	28.300	14.900	32.800

Beam Maximum Moments

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`in)	d (ft)	Max Mz (kip`in)
1	1	9.292	1:DL	Max +ve	0.000	0.000	9.292	2.331
				Max -ve	0.000	0.000	3.097	-1.282
			2:DL EXTERN/	Max +ve	0.000	0.000	9.292	2.438
				Max -ve	0.000	0.000	3.871	-1.810
			3:LL	Max +ve	0.000	0.000	9.292	3.196
				Max -ve	0.000	0.000	3.871	-2.025
			4:WL NEG	Max +ve	0.000	0.000	3.871	2.998
				Max -ve	0.000	0.000	9.292	-5.566
			5:WL B	Max +ve	0.000	0.000	9.292	0.668
				Max -ve	0.000	0.000	3.871	-0.450
			100:DL+LL	Max +ve	0.000	0.000	9.292	7.965
				Max -ve	0.000	0.000	3.871	-5.105
			101:DL+0.6WL	Max +ve	0.000	0.000	9.292	1.429
				Max -ve	0.000	0.000	2.323	-1.414
			102:DL+0.6WL	Max +ve	0.000	0.000	9.292	5.169
				Max -ve	0.000	0.000	3.871	-3.350
			103:DL+0.75LL	Max +ve	0.000	0.000	9.292	4.661
				Max -ve	0.000	0.000	3.871	-3.249
			104:DL+0.75LL	Max +ve	0.000	0.000	9.292	7.466
				Max -ve	0.000	0.000	3.871	-4.801
			105:0.6DL+0.6'	Max +ve	0.000	0.000	5.420	0.082
				Max -ve	0.000	0.000	0.000	-1.267
			106:0.6DL+0.6'	Max +ve	0.000	0.000	9.292	3.262
				Max -ve	0.000	0.000	3.871	-2.118
2	2	7.479	1:DL	Max +ve	0.000	0.000	0.000	1.243
				Max -ve	0.000	0.000	4.363	-0.589

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`in)	d (ft)	Max Mz (kip`in)
			2:DL EXTERN/	Max +ve	0.000	0.000	7.479	4.239
				Max -ve	0.000	0.000	1.870	-0.005
			3:LL	Max +ve	0.000	0.000	7.479	3.102
				Max -ve	0.000	0.000	3.116	-0.390
			4:WL NEG	Max +ve	0.000	0.000	4.363	2.119
				Max -ve	0.000	0.000	0.000	-4.370
			5:WL B	Max +ve	0.000	0.000	7.479	0.557
				Max -ve	0.000	0.000	3.740	-0.124
			100:DL+LL	Max +ve	0.000	0.000	7.479	7.886
				Max -ve	0.000	0.000	3.116	-0.739
			101:DL+0.6WL	Max +ve	0.000	0.000	7.479	4.288
				Max -ve	0.000	0.000	0.000	-0.704
			102:DL+0.6WL	Max +ve	0.000	0.000	7.479	5.118
				Max -ve	0.000	0.000	3.116	-0.422
			103:DL+0.75LL	Max +ve	0.000	0.000	7.479	6.739
				Max -ve	0.000	0.000	2.493	-0.108
			104:DL+0.75LL	Max +ve	0.000	0.000	7.479	7.361
				Max -ve	0.000	0.000	3.116	-0.696
			105:0.6DL+0.6'	Max +ve	0.000	0.000	7.479	2.375
				Max -ve	0.000	0.000	0.000	-1.471
			106:0.6DL+0.6'	Max +ve	0.000	0.000	7.479	3.205
				Max -ve	0.000	0.000	3.116	-0.283
3	3	4.387	1:DL	Max +ve	0.000	0.000	4.387	0.399
				Max -ve	0.000	0.000	2.193	-0.106
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	13.900
				Max -ve	0.000	0.000	4.387	-17.182
			3:LL	Max +ve	0.000	0.000	0.000	8.058
				Max -ve	0.000	0.000	4.387	-9.027
			4:WL NEG	Max +ve	0.000	0.000	0.000	3.427
				Max -ve	0.000	0.000	4.387	-7.413
			5:WL B	Max +ve	0.000	0.000	0.000	1.611
				Max -ve	0.000	0.000	4.387	-1.459
			100:DL+LL	Max +ve	0.000	0.000	0.000	22.356
				Max -ve	0.000	0.000	4.387	-25.810
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	16.354
				Max -ve	0.000	0.000	4.387	-21.231
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	15.264
				Max -ve	0.000	0.000	4.387	-17.658
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	21.883
				Max -ve	0.000	0.000	4.387	-26.889
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	21.066
				Max -ve	0.000	0.000	4.387	-24.210
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	10.635
				Max -ve	0.000	0.000	4.387	-14.518
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	9.545

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kipin)	d (ft)	Max Mz (kip in)
				Max -ve	0.000	0.000	4.387	-10.945
4	4	3.643	1:DL	Max +ve	0.000	0.000	3.643	0.345
				Max -ve	0.000	0.000	1.518	-0.092
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	15.940
				Max -ve	0.000	0.000	3.643	-13.434
			3:LL	Max +ve	0.000	0.000	0.000	8.941
				Max -ve	0.000	0.000	3.643	-6.955
			4:WL NEG	Max +ve	0.000	0.000	0.000	3.679
				Max -ve	0.000	0.000	3.643	-5.332
			5:WL B	Max +ve	0.000	0.000	0.000	3.415
				Max -ve	0.000	0.000	3.643	-1.861
			100:DL+LL	Max +ve	0.000	0.000	0.000	25.056
				Max -ve	0.000	0.000	3.643	-20.043
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	18.322
				Max -ve	0.000	0.000	3.643	-16.287
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	18.164
				Max -ve	0.000	0.000	3.643	-14.205
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	24.476
				Max -ve	0.000	0.000	3.643	-20.704
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	24.358
				Max -ve	0.000	0.000	3.643	-19.142
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	11.876
				Max -ve	0.000	0.000	3.643	-11.052
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	11.718
				Max -ve	0.000	0.000	3.643	-8.970
5	5	3.644	1:DL	Max +ve	0.000	0.000	0.000	0.345
				Max -ve	0.000	0.000	2.126	-0.091
			2:DL EXTERN/	Max +ve	0.000	0.000	3.644	16.108
				Max -ve	0.000	0.000	0.000	-13.434
			3:LL	Max +ve	0.000	0.000	3.644	9.030
				Max -ve	0.000	0.000	0.000	-6.955
			4:WL NEG	Max +ve	0.000	0.000	3.644	4.206
				Max -ve	0.000	0.000	0.000	-5.332
			5:WL B	Max +ve	0.000	0.000	3.644	2.502
				Max -ve	0.000	0.000	0.000	-1.861
			100:DL+LL	Max +ve	0.000	0.000	3.644	25.316
				Max -ve	0.000	0.000	0.000	-20.043
			101:DL+0.6WL	Max +ve	0.000	0.000	3.644	18.809
				Max -ve	0.000	0.000	0.000	-16.287
			102:DL+0.6WL	Max +ve	0.000	0.000	3.644	17.786
				Max -ve	0.000	0.000	0.000	-14.205
			103:DL+0.75LL	Max +ve	0.000	0.000	3.644	24.951
				Max -ve	0.000	0.000	0.000	-20.704
			104:DL+0.75LL	Max +ve	0.000	0.000	3.644	24.184
				Max -ve	0.000	0.000	0.000	-19.142

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip¯in)	d (ft)	Max Mz (kip¯in)
			105:0.6DL+0.6'	Max +ve	0.000	0.000	3.644	12.295
				Max -ve	0.000	0.000	0.000	-11.052
			106:0.6DL+0.6'	Max +ve	0.000	0.000	3.644	11.272
				Max -ve	0.000	0.000	0.000	-8.970
6	6	4.387	1:DL	Max +ve	0.000	0.000	0.000	0.413
				Max -ve	0.000	0.000	2.193	-0.106
			2:DL EXTERN/	Max +ve	0.000	0.000	4.387	13.685
				Max -ve	0.000	0.000	0.000	-16.918
			3:LL	Max +ve	0.000	0.000	4.387	7.946
				Max -ve	0.000	0.000	0.000	-8.888
			4:WL NEG	Max +ve	0.000	0.000	4.387	2.308
				Max -ve	0.000	0.000	0.000	-6.551
			5:WL B	Max +ve	0.000	0.000	4.387	3.670
				Max -ve	0.000	0.000	0.000	-2.973
			100:DL+LL	Max +ve	0.000	0.000	4.387	22.014
				Max -ve	0.000	0.000	0.000	-25.393
			101:DL+0.6WL	Max +ve	0.000	0.000	4.387	15.453
				Max -ve	0.000	0.000	0.000	-20.435
			102:DL+0.6WL	Max +ve	0.000	0.000	4.387	16.270
				Max -ve	0.000	0.000	0.000	-18.289
			103:DL+0.75LL	Max +ve	0.000	0.000	4.387	21.066
				Max -ve	0.000	0.000	0.000	-26.119
			104:DL+0.75LL	Max +ve	0.000	0.000	4.387	21.679
				Max -ve	0.000	0.000	0.000	-24.509
			105:0.6DL+0.6'	Max +ve	0.000	0.000	4.387	9.826
				Max -ve	0.000	0.000	0.000	-13.833
			106:0.6DL+0.6'	Max +ve	0.000	0.000	4.387	10.643
				Max -ve	0.000	0.000	0.000	-11.687
7	7	7.479	1:DL	Max +ve	0.000	0.000	7.479	1.241
				Max -ve	0.000	0.000	3.116	-0.591
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	4.118
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	3.036
				Max -ve	0.000	0.000	4.363	-0.398
			4:WL NEG	Max +ve	0.000	0.000	3.116	2.549
				Max -ve	0.000	0.000	7.479	-5.676
			5:WL B	Max +ve	0.000	0.000	7.479	3.039
				Max -ve	0.000	0.000	3.740	-1.036
			100:DL+LL	Max +ve	0.000	0.000	0.000	7.697
				Max -ve	0.000	0.000	4.363	-0.765
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.585
				Max -ve	0.000	0.000	7.479	-1.430
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	6.102
				Max -ve	0.000	0.000	4.363	-0.894
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	6.131

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip⁻in)	d (ft)	Max Mz (kip⁻in)
				Max -ve	0.000	0.000	5.609	-0.059
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	8.019
				Max -ve	0.000	0.000	4.363	-1.061
			105:0.6DL+0.6'	Max +ve	0.000	0.000	1.247	1.921
				Max -ve	0.000	0.000	7.479	-2.221
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	4.237
				Max -ve	0.000	0.000	3.740	-0.767
8	8	9.292	1:DL	Max +ve	0.000	0.000	0.000	2.334
				Max -ve	0.000	0.000	6.194	-1.281
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	2.452
				Max -ve	0.000	0.000	5.420	-1.801
			3:LL	Max +ve	0.000	0.000	0.000	3.203
				Max -ve	0.000	0.000	5.420	-2.020
			4:WL NEG	Max +ve	0.000	0.000	5.420	4.056
				Max -ve	0.000	0.000	0.000	-7.471
			5:WL B	Max +ve	0.000	0.000	0.000	4.490
				Max -ve	0.000	0.000	5.420	-2.559
			100:DL+LL	Max +ve	0.000	0.000	0.000	7.989
				Max -ve	0.000	0.000	5.420	-5.090
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.303
				Max -ve	0.000	0.000	9.292	-1.610
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	7.480
				Max -ve	0.000	0.000	5.420	-4.605
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	3.826
				Max -ve	0.000	0.000	5.420	-2.760
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	9.209
				Max -ve	0.000	0.000	5.420	-5.736
			105:0.6DL+0.6'	Max +ve	0.000	0.000	4.646	0.674
				Max -ve	0.000	0.000	9.292	-1.801
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	5.566
				Max -ve	0.000	0.000	5.420	-3.377
9	1	8.667	1:DL	Max +ve	0.000	0.000	8.667	1.996
				Max -ve	0.000	0.000	3.611	-1.425
			2:DL EXTERN/	Max +ve	0.000	0.000	8.667	0.540
				Max -ve	0.000	0.000	3.611	-1.187
			3:LL	Max +ve	0.000	0.000	8.667	0.264
				Max -ve	0.000	0.000	0.000	-1.429
			4:WL NEG	Max +ve	0.000	0.000	0.000	2.583
				Max -ve	0.000	0.000	8.667	-1.375
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.278
			100:DL+LL	Max +ve	0.000	0.000	8.667	2.801
				Max -ve	0.000	0.000	2.889	-3.436
			101:DL+0.6WL	Max +ve	0.000	0.000	8.667	1.711
				Max -ve	0.000	0.000	4.333	-2.096

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`in)	d (ft)	Max Mz (kip`in)
			102:DL+0.6WL	Max +ve	0.000	0.000	8.667	2.525
				Max -ve	0.000	0.000	3.611	-2.714
			103:DL+0.75LL	Max +ve	0.000	0.000	8.667	2.116
				Max -ve	0.000	0.000	3.611	-2.734
			104:DL+0.75LL	Max +ve	0.000	0.000	8.667	2.726
				Max -ve	0.000	0.000	2.889	-3.306
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	1.267
				Max -ve	0.000	0.000	4.333	-1.113
			106:0.6DL+0.6'	Max +ve	0.000	0.000	8.667	1.510
				Max -ve	0.000	0.000	3.611	-1.669
10	10	6.792	1:DL	Max +ve	0.000	0.000	0.000	1.426
				Max -ve	0.000	0.000	4.528	-0.201
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	1.129
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.484
				Max -ve	0.000	0.000	6.792	-0.026
			4:WL NEG	Max +ve	0.000	0.000	6.792	0.245
				Max -ve	0.000	0.000	0.000	-0.327
			5:WL B	Max +ve	0.000	0.000	0.000	0.141
				Max -ve	0.000	0.000	6.792	-0.093
			100:DL+LL	Max +ve	0.000	0.000	0.000	3.039
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	2.359
				Max -ve	0.000	0.000	4.528	-0.002
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	2.640
				Max -ve	0.000	0.000	4.528	-0.044
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	2.771
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	2.982
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	1.337
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	1.618
				Max -ve	0.000	0.000	4.528	-0.030
11	11	8.764	1:DL	Max +ve	0.000	0.000	8.764	1.067
				Max -ve	0.000	0.000	4.382	-0.628
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	5.756
				Max -ve	0.000	0.000	8.764	-8.070
			3:LL	Max +ve	0.000	0.000	0.000	2.768
				Max -ve	0.000	0.000	8.764	-4.612
			4:WL NEG	Max +ve	0.000	0.000	0.000	2.405
				Max -ve	0.000	0.000	8.764	-3.359
			5:WL B	Max +ve	0.000	0.000	0.000	0.378
				Max -ve	0.000	0.000	8.764	-0.567
			100:DL+LL	Max +ve	0.000	0.000	0.000	9.392

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip¯in)	d (ft)	Max Mz (kip¯in)
				Max -ve	0.000	0.000	8.764	-11.614
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	8.067
				Max -ve	0.000	0.000	8.764	-9.018
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	6.851
				Max -ve	0.000	0.000	8.764	-7.342
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	9.782
				Max -ve	0.000	0.000	8.764	-11.973
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	8.870
				Max -ve	0.000	0.000	8.764	-10.716
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	5.418
				Max -ve	0.000	0.000	8.764	-6.217
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	4.201
				Max -ve	0.000	0.000	8.764	-4.541
12	12	3.517	1:DL	Max +ve	0.000	0.000	0.000	0.479
				Max -ve	0.000	0.000		
			2:DL EXTERN/	Max +ve	0.000	0.000	3.517	8.337
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	3.517	4.489
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	3.517	3.109
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	2.788
				Max -ve	0.000	0.000	3.517	-0.088
			100:DL+LL	Max +ve	0.000	0.000	3.517	13.295
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	3.517	10.671
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	10.243
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	3.517	13.572
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	13.092
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	3.517	7.149
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	6.815
				Max -ve	0.000	0.000		
13	13	8.724	1:DL	Max +ve	0.000	0.000	0.000	1.056
				Max -ve	0.000	0.000	4.362	-0.618
			2:DL EXTERN/	Max +ve	0.000	0.000	8.724	5.656
				Max -ve	0.000	0.000	0.000	-7.978
			3:LL	Max +ve	0.000	0.000	8.724	2.717
				Max -ve	0.000	0.000	0.000	-4.559
			4:WL NEG	Max +ve	0.000	0.000	8.724	2.098
				Max -ve	0.000	0.000	0.000	-2.626

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip¯in)	d (ft)	Max Mz (kip¯in)
			5:WL B	Max +ve	0.000	0.000	8.724	0.923
				Max -ve	0.000	0.000	0.000	-1.961
			100:DL+LL	Max +ve	0.000	0.000	8.724	9.226
				Max -ve	0.000	0.000	0.000	-11.481
			101:DL+0.6WL	Max +ve	0.000	0.000	8.724	7.769
				Max -ve	0.000	0.000	0.000	-8.497
			102:DL+0.6WL	Max +ve	0.000	0.000	8.724	7.064
				Max -ve	0.000	0.000	0.000	-8.099
			103:DL+0.75LL	Max +ve	0.000	0.000	8.724	9.492
				Max -ve	0.000	0.000	0.000	-11.523
			104:DL+0.75LL	Max +ve	0.000	0.000	8.724	8.963
				Max -ve	0.000	0.000	0.000	-11.224
			105:0.6DL+0.6'	Max +ve	0.000	0.000	8.724	5.165
				Max -ve	0.000	0.000	0.000	-5.729
			106:0.6DL+0.6'	Max +ve	0.000	0.000	8.724	4.460
				Max -ve	0.000	0.000	0.000	-5.330
14	14	6.832	1:DL	Max +ve	0.000	0.000	6.832	1.442
				Max -ve	0.000	0.000	2.277	-0.215
			2:DL EXTERN/	Max +ve	0.000	0.000	6.832	1.182
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	6.832	0.511
				Max -ve	0.000	0.000	0.000	-0.063
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.206
				Max -ve	0.000	0.000	6.832	-0.455
			5:WL B	Max +ve	0.000	0.000	6.832	0.429
				Max -ve	0.000	0.000	0.000	-0.050
			100:DL+LL	Max +ve	0.000	0.000	6.832	3.135
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	6.832	2.351
				Max -ve	0.000	0.000	2.277	-0.087
			102:DL+0.6WL	Max +ve	0.000	0.000	6.832	2.881
				Max -ve	0.000	0.000	1.708	-0.019
			103:DL+0.75LL	Max +ve	0.000	0.000	6.832	2.802
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	6.832	3.200
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	6.832	1.301
				Max -ve	0.000	0.000	2.277	-0.056
			106:0.6DL+0.6'	Max +ve	0.000	0.000	6.832	1.832
				Max -ve	0.000	0.000		
15	15	8.667	1:DL	Max +ve	0.000	0.000	0.000	2.000
				Max -ve	0.000	0.000	5.056	-1.423
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.563
				Max -ve	0.000	0.000	5.056	-1.181
			3:LL	Max +ve	0.000	0.000	0.000	0.277

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip˜in)	d (ft)	Max Mz (kip˜in)
				Max -ve	0.000	0.000	8.667	-1.432
			4:WL NEG	Max +ve	0.000	0.000	8.667	3.477
				Max -ve	0.000	0.000	0.000	-1.819
			5:WL B	Max +ve	0.000	0.000	0.000	0.885
				Max -ve	0.000	0.000	8.667	-2.069
			100:DL+LL	Max +ve	0.000	0.000	0.000	2.840
				Max -ve	0.000	0.000	5.778	-3.428
			101:DL+0.6WL	Max +ve	0.000	0.000	8.667	1.610
				Max -ve	0.000	0.000	4.334	-1.950
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.094
				Max -ve	0.000	0.000	5.778	-3.217
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	1.952
				Max -ve	0.000	0.000	5.056	-2.572
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	3.169
				Max -ve	0.000	0.000	5.778	-3.701
			105:0.6DL+0.6'	Max +ve	0.000	0.000	8.667	1.801
				Max -ve	0.000	0.000	3.611	-1.026
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	2.069
				Max -ve	0.000	0.000	6.500	-2.199
16	10	5.238	1:DL	Max +ve	0.000	0.000	5.238	0.327
				Max -ve	0.000	0.000	0.000	-0.570
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.589
				Max -ve	0.000	0.000	5.238	-0.724
			3:LL	Max +ve	0.000	0.000	0.000	0.219
				Max -ve	0.000	0.000	5.238	-0.158
			4:WL NEG	Max +ve	0.000	0.000	0.000	1.047
				Max -ve	0.000	0.000	5.238	-1.195
			5:WL B	Max +ve	0.000	0.000	0.000	0.160
				Max -ve	0.000	0.000	5.238	-0.164
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.239
				Max -ve	0.000	0.000	5.238	-0.555
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.648
				Max -ve	0.000	0.000	5.238	-1.114
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.115
				Max -ve	0.000	0.000	5.238	-0.496
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.655
				Max -ve	0.000	0.000	5.238	-1.053
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.256
				Max -ve	0.000	0.000	5.238	-0.589
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.640
				Max -ve	0.000	0.000	5.238	-0.955
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.107
				Max -ve	0.000	0.000	5.238	-0.337
17	2	5.360	1:DL	Max +ve	0.000	0.000	0.000	0.761
				Max -ve	0.000	0.000	2.680	-0.260

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip⁻in)	d (ft)	Max Mz (kip⁻in)
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	2.487
				Max -ve	0.000	0.000	5.360	-1.587
			3:LL	Max +ve	0.000	0.000	0.000	1.583
				Max -ve	0.000	0.000	5.360	-0.943
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.360	-0.247
			5:WL B	Max +ve	0.000	0.000	0.000	0.450
				Max -ve	0.000	0.000	5.360	-0.393
			100:DL+LL	Max +ve	0.000	0.000	0.000	4.832
				Max -ve	0.000	0.000	5.360	-2.004
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.248
				Max -ve	0.000	0.000	5.360	-1.208
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.518
				Max -ve	0.000	0.000	5.360	-1.296
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	4.435
				Max -ve	0.000	0.000	5.360	-1.879
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	4.638
				Max -ve	0.000	0.000	5.360	-1.945
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	1.948
				Max -ve	0.000	0.000	5.360	-0.784
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	2.219
				Max -ve	0.000	0.000	5.360	-0.872
18	11	4.238	1:DL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	2.472	-0.238
			2:DL EXTERN/	Max +ve	0.000	0.000	4.238	9.662
				Max -ve	0.000	0.000	0.000	-7.057
			3:LL	Max +ve	0.000	0.000	4.238	4.956
				Max -ve	0.000	0.000	0.000	-3.737
			4:WL NEG	Max +ve	0.000	0.000	4.238	4.252
				Max -ve	0.000	0.000	0.000	-2.406
			5:WL B	Max +ve	0.000	0.000	4.238	1.054
				Max -ve	0.000	0.000	0.000	-0.864
			100:DL+LL	Max +ve	0.000	0.000	4.238	14.470
				Max -ve	0.000	0.000	0.000	-10.890
			101:DL+0.6WL	Max +ve	0.000	0.000	4.238	12.065
				Max -ve	0.000	0.000	0.000	-8.596
			102:DL+0.6WL	Max +ve	0.000	0.000	4.238	10.146
				Max -ve	0.000	0.000	0.000	-7.671
			103:DL+0.75LL	Max +ve	0.000	0.000	4.238	15.144
				Max -ve	0.000	0.000	0.000	-11.038
			104:DL+0.75LL	Max +ve	0.000	0.000	4.238	13.705
				Max -ve	0.000	0.000	0.000	-10.344
			105:0.6DL+0.6'	Max +ve	0.000	0.000	4.238	8.260
				Max -ve	0.000	0.000	0.000	-5.735
			106:0.6DL+0.6'	Max +ve	0.000	0.000	4.238	6.341

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip⁻in)	d (ft)	Max Mz (kip⁻in)
				Max -ve	0.000	0.000	0.000	-4.810
19	4	1.273	1:DL	Max +ve	0.000	0.000	0.000	0.224
				Max -ve	0.000	0.000	1.273	-0.588
			2:DL EXTERN/	Max +ve	0.000	0.000	1.273	16.161
				Max -ve	0.000	0.000	0.000	-33.122
			3:LL	Max +ve	0.000	0.000	1.273	8.968
				Max -ve	0.000	0.000	0.000	-17.968
			4:WL NEG	Max +ve	0.000	0.000	1.273	4.922
				Max -ve	0.000	0.000	0.000	-11.092
			5:WL B	Max +ve	0.000	0.000	1.273	3.355
				Max -ve	0.000	0.000	0.000	-4.874
			100:DL+LL	Max +ve	0.000	0.000	1.273	24.541
				Max -ve	0.000	0.000	0.000	-50.867
			101:DL+0.6WL	Max +ve	0.000	0.000	1.273	18.526
				Max -ve	0.000	0.000	0.000	-39.553
			102:DL+0.6WL	Max +ve	0.000	0.000	1.273	17.586
				Max -ve	0.000	0.000	0.000	-35.822
			103:DL+0.75LL	Max +ve	0.000	0.000	1.273	24.514
				Max -ve	0.000	0.000	0.000	-51.366
			104:DL+0.75LL	Max +ve	0.000	0.000	1.273	23.809
				Max -ve	0.000	0.000	0.000	-48.568
			105:0.6DL+0.6	Max +ve	0.000	0.000	1.273	12.297
				Max -ve	0.000	0.000	0.000	-26.394
			106:0.6DL+0.6	Max +ve	0.000	0.000	1.273	11.356
				Max -ve	0.000	0.000	0.000	-22.663
20	13	1.273	1:DL	Max +ve	0.000	0.000	1.273	0.236
				Max -ve	0.000	0.000	0.000	-0.587
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	16.315
				Max -ve	0.000	0.000	1.273	-33.026
			3:LL	Max +ve	0.000	0.000	0.000	9.048
				Max -ve	0.000	0.000	1.273	-17.918
			4:WL NEG	Max +ve	0.000	0.000	0.000	5.735
				Max -ve	0.000	0.000	1.273	-10.757
			5:WL B	Max +ve	0.000	0.000	0.000	1.873
				Max -ve	0.000	0.000	1.273	-5.475
			100:DL+LL	Max +ve	0.000	0.000	0.000	24.776
				Max -ve	0.000	0.000	1.273	-50.709
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	19.168
				Max -ve	0.000	0.000	1.273	-39.244
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	16.852
				Max -ve	0.000	0.000	1.273	-36.075
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	25.094
				Max -ve	0.000	0.000	1.273	-51.070
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	23.357
				Max -ve	0.000	0.000	1.273	-48.693

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip-in)	d (ft)	Max Mz (kip-in)
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	12.877
				Max -ve	0.000	0.000	1.273	-26.128
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	10.560
				Max -ve	0.000	0.000	1.273	-22.959
21	7	4.226	1:DL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.761	-0.233
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	9.567
				Max -ve	0.000	0.000	4.226	-7.036
			3:LL	Max +ve	0.000	0.000	0.000	4.909
				Max -ve	0.000	0.000	4.226	-3.732
			4:WL NEG	Max +ve	0.000	0.000	0.000	4.101
				Max -ve	0.000	0.000	4.226	-2.191
			5:WL B	Max +ve	0.000	0.000	0.000	1.268
				Max -ve	0.000	0.000	4.226	-1.261
			100:DL+LL	Max +ve	0.000	0.000	0.000	14.317
				Max -ve	0.000	0.000	4.226	-10.855
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	11.868
				Max -ve	0.000	0.000	4.226	-8.437
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	10.169
				Max -ve	0.000	0.000	4.226	-7.880
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	14.935
				Max -ve	0.000	0.000	4.226	-10.908
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	13.660
				Max -ve	0.000	0.000	4.226	-10.490
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	8.105
				Max -ve	0.000	0.000	4.226	-5.588
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	6.405
				Max -ve	0.000	0.000	4.226	-5.031
22	14	5.420	1:DL	Max +ve	0.000	0.000	5.420	0.778
				Max -ve	0.000	0.000	2.710	-0.274
			2:DL EXTERN/	Max +ve	0.000	0.000	5.420	2.474
				Max -ve	0.000	0.000	0.000	-1.605
			3:LL	Max +ve	0.000	0.000	5.420	1.575
				Max -ve	0.000	0.000	0.000	-0.953
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.298
			5:WL B	Max +ve	0.000	0.000	5.420	0.878
				Max -ve	0.000	0.000	0.000	-0.288
			100:DL+LL	Max +ve	0.000	0.000	5.420	4.827
				Max -ve	0.000	0.000	0.000	-2.034
			101:DL+0.6WL	Max +ve	0.000	0.000	5.420	3.120
				Max -ve	0.000	0.000	0.000	-1.261
			102:DL+0.6WL	Max +ve	0.000	0.000	5.420	3.779
				Max -ve	0.000	0.000	0.000	-1.254
			103:DL+0.75LL	Max +ve	0.000	0.000	5.420	4.334

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip¯in)	d (ft)	Max Mz (kip¯in)
				Max -ve	0.000	0.000	0.000	-1.930
			104:DL+0.75LL	Max +ve	0.000	0.000	5.420	4.828
				Max -ve	0.000	0.000	0.000	-1.925
			105:0.6DL+0.6'	Max +ve	0.000	0.000	5.420	1.820
				Max -ve	0.000	0.000	0.000	-0.828
			106:0.6DL+0.6'	Max +ve	0.000	0.000	5.420	2.478
				Max -ve	0.000	0.000	0.000	-0.821
23	8	5.238	1:DL	Max +ve	0.000	0.000	0.000	0.315
				Max -ve	0.000	0.000	5.238	-0.559
			2:DL EXTERN/	Max +ve	0.000	0.000	5.238	0.619
				Max -ve	0.000	0.000	0.000	-0.757
			3:LL	Max +ve	0.000	0.000	5.238	0.234
				Max -ve	0.000	0.000	0.000	-0.175
			4:WL NEG	Max +ve	0.000	0.000	5.238	1.364
				Max -ve	0.000	0.000	0.000	-1.575
			5:WL B	Max +ve	0.000	0.000	0.000	0.573
				Max -ve	0.000	0.000	5.238	-0.456
			100:DL+LL	Max +ve	0.000	0.000	5.238	0.295
				Max -ve	0.000	0.000	0.000	-0.616
			101:DL+0.6WL	Max +ve	0.000	0.000	5.238	0.879
				Max -ve	0.000	0.000	0.000	-1.387
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.056	-0.315
			103:DL+0.75LL	Max +ve	0.000	0.000	5.238	0.850
				Max -ve	0.000	0.000	0.000	-1.281
			104:DL+0.75LL	Max +ve	0.000	0.000	5.238	0.031
				Max -ve	0.000	0.000	1.310	-0.344
			105:0.6DL+0.6'	Max +ve	0.000	0.000	5.238	0.855
				Max -ve	0.000	0.000	0.000	-1.210
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.079
				Max -ve	0.000	0.000	4.802	-0.239

Beam Maximum Shear Forces

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
1	1	9.292	1:DL	Max +ve	0.000	0.000	0.000	0.060
				Max -ve	0.000	0.000	9.292	-0.103
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.099
				Max -ve	0.000	0.000	9.292	-0.133
			3:LL	Max +ve	0.000	0.000	0.000	0.139
				Max -ve	0.000	0.000	9.292	-0.170
			4:WL NEG	Max +ve	0.000	0.000	9.292	0.279
				Max -ve	0.000	0.000	0.000	-0.225
			5:WL B	Max +ve	0.000	0.000	0.000	0.029
				Max -ve	0.000	0.000	9.292	-0.036
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.297
				Max -ve	0.000	0.000	9.292	-0.406
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.024
				Max -ve	0.000	0.000	9.292	-0.069
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.176
				Max -ve	0.000	0.000	9.292	-0.257
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.161
				Max -ve	0.000	0.000	9.292	-0.238
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.276
				Max -ve	0.000	0.000	9.292	-0.380
			105:0.6DL+0.6	Max +ve	0.000	0.000	9.292	0.026
				Max -ve	0.000	0.000	0.000	-0.040
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.113
				Max -ve	0.000	0.000	9.292	-0.163
2	2	7.479	1:DL	Max +ve	0.000	0.000	0.000	0.073
				Max -ve	0.000	0.000	7.479	-0.058
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.054
				Max -ve	0.000	0.000	7.479	-0.133
			3:LL	Max +ve	0.000	0.000	0.000	0.110
				Max -ve	0.000	0.000	7.479	-0.139
			4:WL NEG	Max +ve	0.000	0.000	7.479	0.163
				Max -ve	0.000	0.000	0.000	-0.242
			5:WL B	Max +ve	0.000	0.000	0.000	0.024
				Max -ve	0.000	0.000	7.479	-0.028
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.236
				Max -ve	0.000	0.000	7.479	-0.330
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	7.479	-0.093
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.141
				Max -ve	0.000	0.000	7.479	-0.208
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.100
				Max -ve	0.000	0.000	7.479	-0.222
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.220
				Max -ve	0.000	0.000	7.479	-0.308

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			105:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.069
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.091
				Max -ve	0.000	0.000	7.479	-0.131
3	3	4.387	1:DL	Max +ve	0.000	0.000	0.000	0.038
				Max -ve	0.000	0.000	4.387	-0.038
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.645
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.398
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	4.387	0.325
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.074
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	1.081
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.736
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.728
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	1.021
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	1.015
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	4.387	0.493
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.454
				Max -ve	0.000	0.000		
4	4	3.643	1:DL	Max +ve	0.000	0.000	0.000	0.028
				Max -ve	0.000	0.000	3.643	-0.036
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.717
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.424
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	3.643	0.305
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.134
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	1.170
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.810
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.825
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	1.112

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	1.124
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	3.643	0.537
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.527
				Max -ve	0.000	0.000		
5	5	3.644	1:DL	Max +ve	0.000	0.000	0.000	0.036
				Max -ve	0.000	0.000	3.644	-0.028
			2:DL EXTERN/	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-0.721
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-0.426
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.351
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-0.181
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-1.175
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.805
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-0.858
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-1.107
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-1.150
			105:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.567
			106:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	3.644	-0.558
6	6	4.387	1:DL	Max +ve	0.000	0.000	0.000	0.039
				Max -ve	0.000	0.000	4.387	-0.038
			2:DL EXTERN/	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.636
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.393
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.329
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.224
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-1.067
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.685

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.808
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.972
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-1.069
			105:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.490
			106:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.387	-0.539
7	7	7.479	1:DL	Max +ve	0.000	0.000	0.000	0.058
				Max -ve	0.000	0.000	7.479	-0.073
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.131
				Max -ve	0.000	0.000	7.479	-0.056
			3:LL	Max +ve	0.000	0.000	0.000	0.138
				Max -ve	0.000	0.000	7.479	-0.111
			4:WL NEG	Max +ve	0.000	0.000	7.479	0.316
				Max -ve	0.000	0.000	0.000	-0.230
			5:WL B	Max +ve	0.000	0.000	0.000	0.160
				Max -ve	0.000	0.000	7.479	-0.175
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.327
				Max -ve	0.000	0.000	7.479	-0.239
			101:DL+0.6WL	Max +ve	0.000	0.000	7.479	0.061
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.285
				Max -ve	0.000	0.000	7.479	-0.233
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.189
				Max -ve	0.000	0.000	7.479	-0.069
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.364
				Max -ve	0.000	0.000	7.479	-0.290
			105:0.6DL+0.6	Max +ve	0.000	0.000	7.479	0.113
				Max -ve	0.000	0.000	0.000	-0.025
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.209
				Max -ve	0.000	0.000	7.479	-0.182
8	8	9.292	1:DL	Max +ve	0.000	0.000	0.000	0.103
				Max -ve	0.000	0.000	9.292	-0.060
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.133
				Max -ve	0.000	0.000	9.292	-0.099
			3:LL	Max +ve	0.000	0.000	0.000	0.171
				Max -ve	0.000	0.000	9.292	-0.139
			4:WL NEG	Max +ve	0.000	0.000	9.292	0.304
				Max -ve	0.000	0.000	0.000	-0.375
			5:WL B	Max +ve	0.000	0.000	0.000	0.230
				Max -ve	0.000	0.000	9.292	-0.186
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.406

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000	9.292	-0.297
			101:DL+0.6WL	Max +ve	0.000	0.000	9.292	0.024
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.374
				Max -ve	0.000	0.000	9.292	-0.270
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.195
				Max -ve	0.000	0.000	9.292	-0.126
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.467
				Max -ve	0.000	0.000	9.292	-0.346
			105:0.6DL+0.6'	Max +ve	0.000	0.000	9.292	0.087
				Max -ve	0.000	0.000	0.000	-0.084
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.279
				Max -ve	0.000	0.000	9.292	-0.207
9	1	8.667	1:DL	Max +ve	0.000	0.000	0.000	0.072
				Max -ve	0.000	0.000	8.667	-0.109
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.033
				Max -ve	0.000	0.000	8.667	-0.054
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.016
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.038
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.002
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.089
				Max -ve	0.000	0.000	8.667	-0.179
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.128
				Max -ve	0.000	0.000	8.667	-0.140
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.104
				Max -ve	0.000	0.000	8.667	-0.165
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.110
				Max -ve	0.000	0.000	8.667	-0.158
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.092
				Max -ve	0.000	0.000	8.667	-0.177
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.086
				Max -ve	0.000	0.000	8.667	-0.075
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.062
				Max -ve	0.000	0.000	8.667	-0.099
10	10	6.792	1:DL	Max +ve	0.000	0.000	0.000	0.061
				Max -ve	0.000	0.000	6.792	-0.032
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.032
				Max -ve	0.000	0.000	6.792	-0.012
			3:LL	Max +ve	0.000	0.000	0.000	0.006
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.007

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			5:WL B	Max +ve	0.000	0.000	0.000	0.003
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.100
				Max -ve	0.000	0.000	6.792	-0.037
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.089
				Max -ve	0.000	0.000	6.792	-0.048
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.095
				Max -ve	0.000	0.000	6.792	-0.042
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.095
				Max -ve	0.000	0.000	6.792	-0.042
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.099
				Max -ve	0.000	0.000	6.792	-0.038
			105:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.052
				Max -ve	0.000	0.000	6.792	-0.030
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.058
				Max -ve	0.000	0.000	6.792	-0.025
11	11	8.764	1:DL	Max +ve	0.000	0.000	0.000	0.059
				Max -ve	0.000	0.000	8.764	-0.063
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.160
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.070
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.055
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.009
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.289
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.252
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.225
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.297
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.276
				Max -ve	0.000	0.000		
			105:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.164
				Max -ve	0.000	0.000		
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.137
				Max -ve	0.000	0.000		
12	12	3.517	1:DL	Max +ve	0.000	0.000	0.000	0.037
				Max -ve	0.000	0.000	3.517	-0.037
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.012
				Max -ve	0.000	0.000	3.517	-0.023
			3:LL	Max +ve	0.000	0.000		

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000	0.000	-0.003
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.037
			5:WL B	Max +ve	0.000	0.000	0.000	0.068
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.046
				Max -ve	0.000	0.000	3.517	-0.063
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.027
				Max -ve	0.000	0.000	3.517	-0.082
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.090
				Max -ve	0.000	0.000	3.517	-0.019
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.030
				Max -ve	0.000	0.000	3.517	-0.079
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.077
				Max -ve	0.000	0.000	3.517	-0.032
			105:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.007
				Max -ve	0.000	0.000	3.517	-0.058
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.070
				Max -ve	0.000	0.000		
13	13	8.724	1:DL	Max +ve	0.000	0.000	0.000	0.062
				Max -ve	0.000	0.000	8.724	-0.058
			2:DL EXTERN/	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.159
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.069
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.045
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.028
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.287
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.244
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.234
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.289
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.282
			105:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.157
			106:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	8.724	-0.147
14	14	6.832	1:DL	Max +ve	0.000	0.000	0.000	0.032
				Max -ve	0.000	0.000	6.832	-0.062

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.011
				Max -ve	0.000	0.000	6.832	-0.034
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.007
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.008
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.006
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.036
				Max -ve	0.000	0.000	6.832	-0.103
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.048
				Max -ve	0.000	0.000	6.832	-0.091
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.040
				Max -ve	0.000	0.000	6.832	-0.099
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.042
				Max -ve	0.000	0.000	6.832	-0.097
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.035
				Max -ve	0.000	0.000	6.832	-0.104
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.031
				Max -ve	0.000	0.000	6.832	-0.053
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.022
				Max -ve	0.000	0.000	6.832	-0.061
15	15	8.667	1:DL	Max +ve	0.000	0.000	0.000	0.109
				Max -ve	0.000	0.000	8.667	-0.072
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.054
				Max -ve	0.000	0.000	8.667	-0.033
			3:LL	Max +ve	0.000	0.000	0.000	0.016
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.051
			5:WL B	Max +ve	0.000	0.000	0.000	0.028
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.180
				Max -ve	0.000	0.000	8.667	-0.089
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.133
				Max -ve	0.000	0.000	8.667	-0.136
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.181
				Max -ve	0.000	0.000	8.667	-0.088
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.153
				Max -ve	0.000	0.000	8.667	-0.116
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.189
				Max -ve	0.000	0.000	8.667	-0.080
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.068
				Max -ve	0.000	0.000	8.667	-0.094
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.115

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000	8.667	-0.046
16	10	5.238	1:DL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.238	-0.024
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.021
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.006
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.036
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.005
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.022
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.038
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.020
				Max -ve	0.000	0.000	5.238	-0.000
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.037
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.023
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.031
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.013
				Max -ve	0.000	0.000		
17	2	5.360	1:DL	Max +ve	0.000	0.000	0.000	0.060
				Max -ve	0.000	0.000	5.360	-0.053
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.063
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.039
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.004
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.013
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.163
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.126
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.131
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.154
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.159
				Max -ve	0.000	0.000		

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.076
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.082
				Max -ve	0.000	0.000		
18	11	4.238	1:DL	Max +ve	0.000	0.000	0.000	0.010
				Max -ve	0.000	0.000	4.238	-0.008
			2:DL EXTERN/	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.329
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.171
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.131
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.038
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.508
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.415
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.359
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.524
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.482
			105:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.281
			106:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	4.238	-0.225
19	4	1.273	1:DL	Max +ve	0.000	0.000	0.000	0.066
				Max -ve	0.000	0.000		
			2:DL EXTERN/	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-3.226
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-1.763
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-1.048
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.539
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-4.950
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-3.815
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-3.510
			103:DL+0.75LL	Max +ve	0.000	0.000		

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000	1.273	-4.981
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-4.751
			105:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-2.541
			106:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-2.235
20	13	1.273	1:DL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	1.273	-0.067
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	3.230
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	1.765
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	0.000	1.080
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.481
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	4.955
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.837
				Max -ve	0.000	0.000		
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	3.478
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	4.999
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	4.730
				Max -ve	0.000	0.000		
			105:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	2.561
				Max -ve	0.000	0.000		
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	2.202
				Max -ve	0.000	0.000		
21	7	4.226	1:DL	Max +ve	0.000	0.000	0.000	0.007
				Max -ve	0.000	0.000	4.226	-0.010
			2:DL EXTERN/	Max +ve	0.000	0.000	0.000	0.327
				Max -ve	0.000	0.000		
			3:LL	Max +ve	0.000	0.000	0.000	0.170
				Max -ve	0.000	0.000		
			4:WL NEG	Max +ve	0.000	0.000	0.000	0.124
				Max -ve	0.000	0.000		
			5:WL B	Max +ve	0.000	0.000	0.000	0.050
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000	0.000	0.505
				Max -ve	0.000	0.000		
			101:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.409
				Max -ve	0.000	0.000		

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.364
				Max -ve	0.000	0.000		
			103:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.518
				Max -ve	0.000	0.000		
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.485
				Max -ve	0.000	0.000		
			105:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.275
				Max -ve	0.000	0.000		
			106:0.6DL+0.6	Max +ve	0.000	0.000	0.000	0.231
				Max -ve	0.000	0.000		
22	14	5.420	1:DL	Max +ve	0.000	0.000	0.000	0.053
				Max -ve	0.000	0.000	5.420	-0.061
			2:DL EXTERN	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.063
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.039
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.001
			5:WL B	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.018
			100:DL+LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.162
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.124
			102:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.134
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.153
			104:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.161
			105:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.075
			106:0.6DL+0.6	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.420	-0.085
23	8	5.238	1:DL	Max +ve	0.000	0.000	0.000	0.024
				Max -ve	0.000	0.000		
			2:DL EXTERN	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.022
			3:LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.007
			4:WL NEG	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	0.000	-0.047
			5:WL B	Max +ve	0.000	0.000	0.000	0.016
				Max -ve	0.000	0.000		
			100:DL+LL	Max +ve	0.000	0.000		

Beam Maximum Shear Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max -ve	0.000	0.000	5.238	-0.024
			101:DL+0.6WL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.238	-0.046
			102:DL+0.6WL	Max +ve	0.000	0.000	0.000	0.012
				Max -ve	0.000	0.000	5.238	-0.008
			103:DL+0.75LL	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.238	-0.044
			104:DL+0.75LL	Max +ve	0.000	0.000	0.000	0.004
				Max -ve	0.000	0.000	5.238	-0.015
			105:0.6DL+0.6'	Max +ve	0.000	0.000		
				Max -ve	0.000	0.000	5.238	-0.039
			106:0.6DL+0.6'	Max +ve	0.000	0.000	0.000	0.011
				Max -ve	0.000	0.000	5.238	-0.001

Beam Maximum Axial Forces

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
1	1	9.292	1:DL	Max +ve	0.000	0.070
				Max -ve	9.292	-0.038
			2:DL EXTERN/	Max +ve	0.000	4.146
				Max -ve		
			3:LL	Max +ve	0.000	2.311
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.405
				Max -ve		
			5:WL B	Max +ve	0.000	0.835
				Max -ve		
			100:DL+LL	Max +ve	0.000	6.528
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	5.059
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	4.717
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	6.582
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	6.326
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	3.373
				Max -ve		
			106:0.6DL+0.6'	Max +ve	0.000	3.031
				Max -ve		
2	2	7.479	1:DL	Max +ve	0.000	0.345

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	4.774
				Max -ve		
			3:LL	Max +ve	0.000	2.670
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.134
				Max -ve		
			5:WL B	Max +ve	0.000	0.926
				Max -ve		
			100:DL+LL	Max +ve	0.000	7.789
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	5.799
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	5.674
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	7.632
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	7.538
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	3.752
				Max -ve		
			106:0.6DL+0.6'	Max +ve	0.000	3.627
				Max -ve		
3	3	4.387	1:DL	Max +ve	0.000	0.366
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	5.908
				Max -ve		
			3:LL	Max +ve	0.000	3.293
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.243
				Max -ve		
			5:WL B	Max +ve	0.000	1.084
				Max -ve		
			100:DL+LL	Max +ve	0.000	9.567
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	7.020
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	6.924
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	9.303
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	9.231
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	4.510
				Max -ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			106:0.6DL+0.6	Max +ve	0.000	4.415
				Max -ve		
4	4	3.643	1:DL	Max +ve	0.000	0.096
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	11.376
				Max -ve		
			3:LL	Max +ve	0.000	6.115
				Max -ve		
			4:WL NEG	Max +ve	0.000	3.459
				Max -ve		
			5:WL B	Max +ve	0.000	1.919
				Max -ve		
			100:DL+LL	Max +ve	0.000	17.587
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	13.548
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	12.623
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	17.615
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	16.922
				Max -ve		
			105:0.6DL+0.6	Max +ve	0.000	8.959
				Max -ve		
			106:0.6DL+0.6	Max +ve	0.000	8.035
				Max -ve		
5	5	3.644	1:DL	Max +ve	3.644	0.096
				Max -ve		
			2:DL EXTERN/	Max +ve	3.644	11.378
				Max -ve		
			3:LL	Max +ve	3.644	6.115
				Max -ve		
			4:WL NEG	Max +ve	0.000	3.489
				Max -ve		
			5:WL B	Max +ve	0.000	1.859
				Max -ve		
			100:DL+LL	Max +ve	3.644	17.589
				Max -ve		
			101:DL+0.6WL	Max +ve	3.644	13.567
				Max -ve		
			102:DL+0.6WL	Max +ve	3.644	12.589
				Max -ve		
			103:DL+0.75LL	Max +ve	3.644	17.630
				Max -ve		
			104:DL+0.75LL	Max +ve	3.644	16.896

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve		
			105:0.6DL+0.6	Max +ve	3.644	8.978
				Max -ve		
			106:0.6DL+0.6	Max +ve	3.644	7.999
				Max -ve		
6	6	4.387	1:DL	Max +ve	4.387	0.369
				Max -ve		
			2:DL EXTERN/	Max +ve	4.387	5.919
				Max -ve		
			3:LL	Max +ve	4.387	3.299
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.178
				Max -ve		
			5:WL B	Max +ve	0.000	1.221
				Max -ve		
			100:DL+LL	Max +ve	4.387	9.586
				Max -ve		
			101:DL+0.6WL	Max +ve	4.387	6.994
				Max -ve		
			102:DL+0.6WL	Max +ve	4.387	7.020
				Max -ve		
			103:DL+0.75LL	Max +ve	4.387	9.291
				Max -ve		
			104:DL+0.75LL	Max +ve	4.387	9.311
				Max -ve		
			105:0.6DL+0.6	Max +ve	4.387	4.479
				Max -ve		
			106:0.6DL+0.6	Max +ve	4.387	4.505
				Max -ve		
7	7	7.479	1:DL	Max +ve	7.479	0.352
				Max -ve		
			2:DL EXTERN/	Max +ve	7.479	4.826
				Max -ve		
			3:LL	Max +ve	7.479	2.701
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.222
				Max -ve		
			5:WL B	Max +ve	0.000	0.771
				Max -ve		
			100:DL+LL	Max +ve	7.479	7.879
				Max -ve		
			101:DL+0.6WL	Max +ve	7.479	5.911
				Max -ve		
			102:DL+0.6WL	Max +ve	7.479	5.640
				Max -ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			103:DL+0.75LL	Max +ve	7.479	7.754
				Max -ve		
			104:DL+0.75LL	Max +ve	7.479	7.551
				Max -ve		
			105:0.6DL+0.6'	Max +ve	7.479	3.840
				Max -ve		
			106:0.6DL+0.6'	Max +ve	7.479	3.569
				Max -ve		
8	8	9.292	1:DL	Max +ve	9.292	0.069
				Max -ve	0.000	-0.039
			2:DL EXTERN/	Max +ve	9.292	4.110
				Max -ve		
			3:LL	Max +ve	9.292	2.292
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.684
				Max -ve		
			5:WL B	Max +ve	0.000	0.245
				Max -ve		
			100:DL+LL	Max +ve	9.292	6.471
				Max -ve		
			101:DL+0.6WL	Max +ve	9.292	5.189
				Max -ve		
			102:DL+0.6WL	Max +ve	9.292	4.326
				Max -ve		
			103:DL+0.75LL	Max +ve	9.292	6.656
				Max -ve		
			104:DL+0.75LL	Max +ve	9.292	6.008
				Max -ve		
			105:0.6DL+0.6'	Max +ve	9.292	3.518
				Max -ve		
			106:0.6DL+0.6'	Max +ve	9.292	2.654
				Max -ve		
9	1	8.667	1:DL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			2:DL EXTERN/	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			3:LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			4:WL NEG	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			5:WL B	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			100:DL+LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			101:DL+0.6WL	Max +ve	0.000	0.000

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve	0.000	0.000
			102:DL+0.6WL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			103:DL+0.75LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			104:DL+0.75LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			105:0.6DL+0.6'	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			106:0.6DL+0.6'	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
10	10	6.792	1:DL	Max +ve	0.000	0.746
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	6.134
				Max -ve		
			3:LL	Max +ve	0.000	3.427
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.330
				Max -ve		
			5:WL B	Max +ve	0.000	1.007
				Max -ve		
			100:DL+LL	Max +ve	0.000	10.306
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	7.677
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	7.483
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	10.048
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	9.903
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	4.926
				Max -ve		
			106:0.6DL+0.6'	Max +ve	0.000	4.732
				Max -ve		
11	11	8.764	1:DL	Max +ve	0.000	0.190
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	4.287
				Max -ve		
			3:LL	Max +ve	0.000	2.282
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.299
				Max -ve		
			5:WL B	Max +ve	0.000	0.800
				Max -ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			100:DL+LL	Max +ve	0.000	6.758
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	5.256
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	4.956
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	6.772
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	6.548
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	3.465
				Max -ve		
			106:0.6DL+0.6'	Max +ve	0.000	3.166
				Max -ve		
12	12	3.517	1:DL	Max +ve	0.000	0.300
				Max -ve		
			2:DL EXTERN/	Max +ve		
				Max -ve	0.000	-1.800
			3:LL	Max +ve		
				Max -ve	0.000	-0.879
			4:WL NEG	Max +ve		
				Max -ve	0.000	-1.147
			5:WL B	Max +ve		
				Max -ve	0.000	-0.123
			100:DL+LL	Max +ve		
				Max -ve	0.000	-2.378
			101:DL+0.6WL	Max +ve		
				Max -ve	0.000	-2.188
			102:DL+0.6WL	Max +ve		
				Max -ve	0.000	-1.573
			103:DL+0.75LL	Max +ve		
				Max -ve	0.000	-2.675
			104:DL+0.75LL	Max +ve		
				Max -ve	0.000	-2.214
			105:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-1.588
			106:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-0.973
13	13	8.724	1:DL	Max +ve	8.724	0.187
				Max -ve		
			2:DL EXTERN/	Max +ve	8.724	4.290
				Max -ve		
			3:LL	Max +ve	0.000	2.284
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.440

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve		
			5:WL B	Max +ve	0.000	0.523
				Max -ve		
			100:DL+LL	Max +ve	8.724	6.760
				Max -ve		
			101:DL+0.6WL	Max +ve	8.724	5.341
				Max -ve		
			102:DL+0.6WL	Max +ve	8.724	4.791
				Max -ve		
			103:DL+0.75LL	Max +ve	8.724	6.837
				Max -ve		
			104:DL+0.75LL	Max +ve	8.724	6.425
				Max -ve		
			105:0.6DL+0.6	Max +ve	8.724	3.550
				Max -ve		
			106:0.6DL+0.6	Max +ve	8.724	3.000
				Max -ve		
14	14	6.832	1:DL	Max +ve	6.832	0.743
				Max -ve		
			2:DL EXTERNA	Max +ve	6.832	6.146
				Max -ve		
			3:LL	Max +ve	0.000	3.433
				Max -ve		
			4:WL NEG	Max +ve	0.000	1.207
				Max -ve		
			5:WL B	Max +ve	0.000	1.273
				Max -ve		
			100:DL+LL	Max +ve	6.832	10.323
				Max -ve		
			101:DL+0.6WL	Max +ve	6.832	7.614
				Max -ve		
			102:DL+0.6WL	Max +ve	6.832	7.653
				Max -ve		
			103:DL+0.75LL	Max +ve	6.832	10.007
				Max -ve		
			104:DL+0.75LL	Max +ve	6.832	10.037
				Max -ve		
			105:0.6DL+0.6	Max +ve	6.832	4.858
				Max -ve		
			106:0.6DL+0.6	Max +ve	6.832	4.898
				Max -ve		
15	15	8.667	1:DL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			2:DL EXTERNA	Max +ve	0.000	0.000
				Max -ve	0.000	0.000

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			3:LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			4:WL NEG	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			5:WL B	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			100:DL+LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			101:DL+0.6WL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			102:DL+0.6WL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			103:DL+0.75LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			104:DL+0.75LL	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			105:0.6DL+0.6'	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
			106:0.6DL+0.6'	Max +ve	0.000	0.000
				Max -ve	0.000	0.000
16	10	5.238	1:DL	Max +ve	0.000	0.538
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	0.660
				Max -ve		
			3:LL	Max +ve	0.000	0.594
				Max -ve		
			4:WL NEG	Max +ve		
				Max -ve	0.000	-0.595
			5:WL B	Max +ve	0.000	0.115
				Max -ve		
			100:DL+LL	Max +ve	0.000	1.792
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	0.840
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	1.267
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	1.375
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	1.695
				Max -ve		
			105:0.6DL+0.6'	Max +ve	0.000	0.361
				Max -ve		
			106:0.6DL+0.6'	Max +ve	0.000	0.788
				Max -ve		
17	2	5.360	1:DL	Max +ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve	0.000	-0.321
			2:DL EXTERN/	Max +ve		
				Max -ve	0.000	-0.645
			3:LL	Max +ve		
				Max -ve	0.000	-0.415
			4:WL NEG	Max +ve	0.000	0.078
				Max -ve		
			5:WL B	Max +ve		
				Max -ve	0.000	-0.058
			100:DL+LL	Max +ve		
				Max -ve	0.000	-1.381
			101:DL+0.6WL	Max +ve		
				Max -ve	0.000	-0.919
			102:DL+0.6WL	Max +ve		
				Max -ve	0.000	-1.001
			103:DL+0.75LL	Max +ve		
				Max -ve	0.000	-1.242
			104:DL+0.75LL	Max +ve		
				Max -ve	0.000	-1.303
			105:0.6DL+0.6	Max +ve		
				Max -ve	0.000	-0.533
			106:0.6DL+0.6	Max +ve		
				Max -ve	0.000	-0.614
18	11	4.238	1:DL	Max +ve	0.000	0.232
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	1.442
				Max -ve		
			3:LL	Max +ve	0.000	0.938
				Max -ve		
			4:WL NEG	Max +ve	0.000	0.024
				Max -ve		
			5:WL B	Max +ve	0.000	0.184
				Max -ve		
			100:DL+LL	Max +ve	0.000	2.612
				Max -ve		
			101:DL+0.6WL	Max +ve	0.000	1.689
				Max -ve		
			102:DL+0.6WL	Max +ve	0.000	1.784
				Max -ve		
			103:DL+0.75LL	Max +ve	0.000	2.389
				Max -ve		
			104:DL+0.75LL	Max +ve	0.000	2.460
				Max -ve		
			105:0.6DL+0.6	Max +ve	0.000	1.019
				Max -ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			106:0.6DL+0.6'	Max +ve	0.000	1.115
				Max -ve		
19	4	1.273	1:DL	Max +ve	0.000	0.219
				Max -ve		
			2:DL EXTERN/	Max +ve		
				Max -ve	0.000	-4.509
			3:LL	Max +ve		
				Max -ve	0.000	-2.332
			4:WL NEG	Max +ve		
				Max -ve	0.000	-1.964
			5:WL B	Max +ve		
				Max -ve	0.000	-0.644
			100:DL+LL	Max +ve		
				Max -ve	0.000	-6.623
			101:DL+0.6WL	Max +ve		
				Max -ve	0.000	-5.469
			102:DL+0.6WL	Max +ve		
				Max -ve	0.000	-4.677
			103:DL+0.75LL	Max +ve		
				Max -ve	0.000	-6.923
			104:DL+0.75LL	Max +ve		
				Max -ve	0.000	-6.329
			105:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-3.752
			106:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-2.961
20	13	1.273	1:DL	Max +ve	0.000	0.221
				Max -ve		
			2:DL EXTERN/	Max +ve		
				Max -ve	0.000	-4.495
			3:LL	Max +ve		
				Max -ve	0.000	-2.325
			4:WL NEG	Max +ve		
				Max -ve	0.000	-2.059
			5:WL B	Max +ve		
				Max -ve	0.000	-0.445
			100:DL+LL	Max +ve		
				Max -ve	0.000	-6.599
			101:DL+0.6WL	Max +ve		
				Max -ve	0.000	-5.509
			102:DL+0.6WL	Max +ve		
				Max -ve	0.000	-4.541
			103:DL+0.75LL	Max +ve		
				Max -ve	0.000	-6.944
			104:DL+0.75LL	Max +ve		

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
				Max -ve	0.000	-6.218
			105:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-3.800
			106:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-2.831
21	7	4.226	1:DL	Max +ve	4.226	0.228
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	1.401
				Max -ve		
			3:LL	Max +ve	0.000	0.914
				Max -ve		
			4:WL NEG	Max +ve		
				Max -ve	0.000	-0.189
			5:WL B	Max +ve	0.000	0.590
				Max -ve		
			100:DL+LL	Max +ve	4.226	2.543
				Max -ve		
			101:DL+0.6WL	Max +ve	4.226	1.515
				Max -ve		
			102:DL+0.6WL	Max +ve	4.226	1.983
				Max -ve		
			103:DL+0.75LL	Max +ve	4.226	2.229
				Max -ve		
			104:DL+0.75LL	Max +ve	4.226	2.580
				Max -ve		
			105:0.6DL+0.6'	Max +ve	4.226	0.864
				Max -ve		
			106:0.6DL+0.6'	Max +ve	4.226	1.331
				Max -ve		
22	14	5.420	1:DL	Max +ve		
				Max -ve	0.000	-0.329
			2:DL EXTERN/	Max +ve		
				Max -ve	0.000	-0.725
			3:LL	Max +ve		
				Max -ve	0.000	-0.461
			4:WL NEG	Max +ve	0.000	0.199
				Max -ve		
			5:WL B	Max +ve		
				Max -ve	0.000	-0.348
			100:DL+LL	Max +ve		
				Max -ve	0.000	-1.515
			101:DL+0.6WL	Max +ve		
				Max -ve	0.000	-0.935
			102:DL+0.6WL	Max +ve		
				Max -ve	0.000	-1.263

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
			103:DL+0.75LL	Max +ve		
				Max -ve	0.000	-1.310
			104:DL+0.75LL	Max +ve		
				Max -ve	0.000	-1.556
			105:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-0.513
			106:0.6DL+0.6'	Max +ve		
				Max -ve	0.000	-0.841
23	8	5.238	1:DL	Max +ve	5.238	0.543
				Max -ve		
			2:DL EXTERN/	Max +ve	0.000	0.710
				Max -ve		
			3:LL	Max +ve	0.000	0.623
				Max -ve		
			4:WL NEG	Max +ve		
				Max -ve	0.000	-0.852
			5:WL B	Max +ve	0.000	0.660
				Max -ve		
			100:DL+LL	Max +ve	5.238	1.877
				Max -ve		
			101:DL+0.6WL	Max +ve	5.238	0.742
				Max -ve		
			102:DL+0.6WL	Max +ve	5.238	1.650
				Max -ve		
			103:DL+0.75LL	Max +ve	5.238	1.338
				Max -ve		
			104:DL+0.75LL	Max +ve	5.238	2.018
				Max -ve		
			105:0.6DL+0.6'	Max +ve	5.238	0.241
				Max -ve		
			106:0.6DL+0.6'	Max +ve	5.238	1.148
				Max -ve		

Beam Displacement Detail Summary

	Beam	L/C	d (ft)	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	21	103:DL+0.75LL	0.423	**0.008**	-0.003	0.000	0.008
Min X	18	103:DL+0.75LL	3.814	**-0.008**	-0.003	0.000	0.008
Max Y	8	4:WL NEG	4.646	0.003	**0.004**	0.000	0.005
Min Y	4	100:DL+LL	3.643	0.000	**-0.027**	0.000	0.027
Max Z	1	1:DL	0.000	0.000	0.000	**0.000**	0.000
Min Z	1	1:DL	0.000	0.000	0.000	**0.000**	0.000
Max Rst	4	100:DL+LL	3.643	0.000	-0.027	0.000	**0.027**

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	7	103:DL+0.75LL	**0.008**	-0.003	0.000	0.008	0.000	0.000	0.000
Min X	3	103:DL+0.75LL	**-0.007**	-0.003	0.000	0.008	0.000	0.000	-0.000
Max Y	7	4:WL NEG	0.002	**0.001**	0.000	0.003	0.000	0.000	0.000
Min Y	5	100:DL+LL	0.000	**-0.027**	0.000	0.027	0.000	0.000	0.000
Max Z	1	1:DL	0.000	0.000	**0.000**	0.000	0.000	0.000	-0.000
Min Z	1	1:DL	0.000	0.000	**0.000**	0.000	0.000	0.000	-0.000
Max rX	1	1:DL	0.000	0.000	0.000	0.000	**0.000**	0.000	-0.000
Min rX	1	1:DL	0.000	0.000	0.000	0.000	**0.000**	0.000	-0.000
Max rY	1	1:DL	0.000	0.000	0.000	0.000	0.000	**0.000**	-0.000
Min rY	1	1:DL	0.000	0.000	0.000	0.000	0.000	**0.000**	-0.000
Max rZ	12	103:DL+0.75LL	-0.000	-0.011	0.000	0.011	0.000	0.000	**0.000**
Min rZ	13	100:DL+LL	0.000	-0.011	0.000	0.011	0.000	0.000	**-0.000**
Max Rst	5	100:DL+LL	0.000	-0.027	0.000	**0.027**	0.000	0.000	0.000

Node Displacements

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
1	1:DL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	2:DL EXTERN/	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	3:LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	4:WL NEG	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	5:WL B	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	100:DL+LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	101:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	102:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	103:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	104:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	105:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	106:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
2	1:DL	0.000	-0.000	0.000	0.000	0.000	0.000	-0.000

Node Displacements Cont...

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
	2:DL EXTERN/	-0.003	-0.001	0.000	0.003	0.000	0.000	0.000
	3:LL	-0.001	-0.000	0.000	0.001	0.000	0.000	0.000
	4:WL NEG	-0.001	0.000	0.000	0.001	0.000	0.000	-0.000
	5:WL B	-0.001	-0.000	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	-0.004	-0.001	0.000	0.004	0.000	0.000	0.000
	101:DL+0.6WL	-0.003	-0.001	0.000	0.003	0.000	0.000	0.000
	102:DL+0.6WL	-0.003	-0.001	0.000	0.003	0.000	0.000	0.000
	103:DL+0.75LL	-0.004	-0.001	0.000	0.004	0.000	0.000	0.000
	104:DL+0.75LL	-0.004	-0.001	0.000	0.004	0.000	0.000	0.000
	105:0.6DL+0.6'	-0.002	-0.001	0.000	0.002	0.000	0.000	0.000
	106:0.6DL+0.6'	-0.002	-0.001	0.000	0.002	0.000	0.000	0.000
3	1:DL	0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	2:DL EXTERN/	-0.005	-0.002	0.000	0.005	0.000	0.000	-0.000
	3:LL	-0.003	-0.001	0.000	0.003	0.000	0.000	-0.000
	4:WL NEG	-0.002	0.000	0.000	0.002	0.000	0.000	-0.000
	5:WL B	-0.001	-0.000	0.000	0.001	0.000	0.000	-0.000
	100:DL+LL	-0.007	-0.003	0.000	0.008	0.000	0.000	-0.000
	101:DL+0.6WL	-0.006	-0.002	0.000	0.006	0.000	0.000	-0.000
	102:DL+0.6WL	-0.005	-0.002	0.000	0.006	0.000	0.000	-0.000
	103:DL+0.75LL	-0.007	-0.003	0.000	0.008	0.000	0.000	-0.000
	104:DL+0.75LL	-0.007	-0.003	0.000	0.008	0.000	0.000	-0.000
	105:0.6DL+0.6'	-0.004	-0.001	0.000	0.004	0.000	0.000	-0.000
	106:0.6DL+0.6'	-0.003	-0.001	0.000	0.004	0.000	0.000	-0.000
4	1:DL	0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	2:DL EXTERN/	-0.001	-0.011	0.000	0.011	0.000	0.000	0.000
	3:LL	-0.000	-0.006	0.000	0.006	0.000	0.000	0.000
	4:WL NEG	0.000	-0.003	0.000	0.003	0.000	0.000	0.000
	5:WL B	-0.001	-0.001	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	-0.001	-0.018	0.000	0.018	0.000	0.000	0.000
	101:DL+0.6WL	-0.000	-0.014	0.000	0.014	0.000	0.000	0.000
	102:DL+0.6WL	-0.001	-0.012	0.000	0.012	0.000	0.000	0.000
	103:DL+0.75LL	-0.001	-0.018	0.000	0.018	0.000	0.000	0.000
	104:DL+0.75LL	-0.001	-0.017	0.000	0.017	0.000	0.000	0.000
	105:0.6DL+0.6'	-0.000	-0.009	0.000	0.009	0.000	0.000	0.000
	106:0.6DL+0.6'	-0.001	-0.008	0.000	0.008	0.000	0.000	0.000
5	1:DL	0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	2:DL EXTERN/	0.000	-0.017	0.000	0.017	0.000	0.000	0.000
	3:LL	0.000	-0.009	0.000	0.009	0.000	0.000	0.000
	4:WL NEG	0.000	-0.005	0.000	0.005	0.000	0.000	0.000
	5:WL B	-0.000	-0.003	0.000	0.003	0.000	0.000	-0.000
	100:DL+LL	0.000	-0.027	0.000	0.027	0.000	0.000	0.000
	101:DL+0.6WL	0.000	-0.020	0.000	0.020	0.000	0.000	0.000
	102:DL+0.6WL	-0.000	-0.019	0.000	0.019	0.000	0.000	-0.000
	103:DL+0.75LL	0.000	-0.027	0.000	0.027	0.000	0.000	0.000
	104:DL+0.75LL	-0.000	-0.026	0.000	0.026	0.000	0.000	-0.000

Node Displacements Cont...

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
	105:0.6DL+0.6'	0.000	-0.013	0.000	0.013	0.000	0.000	0.000
	106:0.6DL+0.6'	-0.000	-0.012	0.000	0.012	0.000	0.000	-0.000
6	1:DL	-0.000	-0.001	0.000	0.001	0.000	0.000	-0.000
	2:DL EXTERN/	0.001	-0.011	0.000	0.011	0.000	0.000	-0.000
	3:LL	0.000	-0.006	0.000	0.006	0.000	0.000	-0.000
	4:WL NEG	0.001	-0.003	0.000	0.003	0.000	0.000	-0.000
	5:WL B	-0.001	-0.003	0.000	0.003	0.000	0.000	-0.000
	100:DL+LL	0.001	-0.018	0.000	0.018	0.000	0.000	-0.000
	101:DL+0.6WL	0.001	-0.013	0.000	0.013	0.000	0.000	-0.000
	102:DL+0.6WL	0.000	-0.013	0.000	0.013	0.000	0.000	-0.000
	103:DL+0.75LL	0.001	-0.017	0.000	0.017	0.000	0.000	-0.000
	104:DL+0.75LL	0.000	-0.017	0.000	0.017	0.000	0.000	-0.000
	105:0.6DL+0.6'	0.001	-0.008	0.000	0.008	0.000	0.000	-0.000
	106:0.6DL+0.6'	-0.000	-0.009	0.000	0.009	0.000	0.000	-0.000
7	1:DL	-0.000	-0.001	0.000	0.001	0.000	0.000	-0.000
	2:DL EXTERN/	0.005	-0.002	0.000	0.005	0.000	0.000	0.000
	3:LL	0.002	-0.001	0.000	0.003	0.000	0.000	0.000
	4:WL NEG	0.002	0.001	0.000	0.003	0.000	0.000	0.000
	5:WL B	-0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	0.007	-0.003	0.000	0.008	0.000	0.000	0.000
	101:DL+0.6WL	0.006	-0.002	0.000	0.006	0.000	0.000	0.000
	102:DL+0.6WL	0.005	-0.003	0.000	0.005	0.000	0.000	0.000
	103:DL+0.75LL	0.008	-0.003	0.000	0.008	0.000	0.000	0.000
	104:DL+0.75LL	0.006	-0.004	0.000	0.007	0.000	0.000	0.000
	105:0.6DL+0.6'	0.004	-0.001	0.000	0.004	0.000	0.000	0.000
	106:0.6DL+0.6'	0.003	-0.002	0.000	0.003	0.000	0.000	0.000
8	1:DL	-0.000	-0.000	0.000	0.000	0.000	0.000	0.000
	2:DL EXTERN/	0.003	-0.001	0.000	0.003	0.000	0.000	-0.000
	3:LL	0.001	-0.000	0.000	0.001	0.000	0.000	-0.000
	4:WL NEG	0.001	0.000	0.000	0.001	0.000	0.000	0.000
	5:WL B	0.000	-0.000	0.000	0.000	0.000	0.000	-0.000
	100:DL+LL	0.004	-0.001	0.000	0.004	0.000	0.000	-0.000
	101:DL+0.6WL	0.003	-0.001	0.000	0.003	0.000	0.000	-0.000
	102:DL+0.6WL	0.002	-0.001	0.000	0.003	0.000	0.000	-0.000
	103:DL+0.75LL	0.004	-0.001	0.000	0.004	0.000	0.000	-0.000
	104:DL+0.75LL	0.003	-0.001	0.000	0.004	0.000	0.000	-0.000
	105:0.6DL+0.6'	0.002	-0.000	0.000	0.002	0.000	0.000	-0.000
	106:0.6DL+0.6'	0.001	-0.001	0.000	0.002	0.000	0.000	-0.000
9	1:DL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	2:DL EXTERN/	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	3:LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	4:WL NEG	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	5:WL B	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	100:DL+LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	101:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Node Displacements Cont...

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
	102:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	103:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	104:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	1:DL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	2:DL EXTERN/	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	3:LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	4:WL NEG	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	5:WL B	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	100:DL+LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	101:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	102:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	103:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	104:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	1:DL	0.000	-0.001	0.000	0.001	0.000	0.000	-0.000
	2:DL EXTERN/	-0.002	-0.002	0.000	0.003	0.000	0.000	-0.000
	3:LL	-0.001	-0.001	0.000	0.002	0.000	0.000	-0.000
	4:WL NEG	-0.001	0.000	0.000	0.001	0.000	0.000	0.000
	5:WL B	-0.001	-0.000	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	-0.003	-0.003	0.000	0.005	0.000	0.000	-0.000
	101:DL+0.6WL	-0.003	-0.002	0.000	0.004	0.000	0.000	-0.000
	102:DL+0.6WL	-0.002	-0.002	0.000	0.003	0.000	0.000	-0.000
	103:DL+0.75LL	-0.004	-0.003	0.000	0.005	0.000	0.000	-0.000
	104:DL+0.75LL	-0.003	-0.003	0.000	0.004	0.000	0.000	-0.000
	105:0.6DL+0.6'	-0.002	-0.001	0.000	0.002	0.000	0.000	-0.000
	106:0.6DL+0.6'	-0.002	-0.001	0.000	0.002	0.000	0.000	-0.000
12	1:DL	0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	2:DL EXTERN/	-0.000	-0.007	0.000	0.007	0.000	0.000	0.000
	3:LL	-0.000	-0.004	0.000	0.004	0.000	0.000	0.000
	4:WL NEG	0.000	-0.002	0.000	0.002	0.000	0.000	0.000
	5:WL B	-0.001	-0.001	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	-0.000	-0.011	0.000	0.011	0.000	0.000	0.000
	101:DL+0.6WL	0.000	-0.009	0.000	0.009	0.000	0.000	0.000
	102:DL+0.6WL	-0.001	-0.008	0.000	0.008	0.000	0.000	0.000
	103:DL+0.75LL	-0.000	-0.011	0.000	0.011	0.000	0.000	0.000
	104:DL+0.75LL	-0.001	-0.010	0.000	0.011	0.000	0.000	0.000
	105:0.6DL+0.6'	0.000	-0.006	0.000	0.006	0.000	0.000	0.000
	106:0.6DL+0.6'	-0.001	-0.005	0.000	0.005	0.000	0.000	0.000
13	1:DL	-0.000	-0.001	0.000	0.001	0.000	0.000	-0.000
	2:DL EXTERN/	0.000	-0.007	0.000	0.007	0.000	0.000	-0.000
	3:LL	0.000	-0.004	0.000	0.004	0.000	0.000	-0.000
	4:WL NEG	0.001	-0.001	0.000	0.001	0.000	0.000	-0.000

Node Displacements Cont...

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
	5:WL B	-0.001	-0.002	0.000	0.002	0.000	0.000	-0.000
	100:DL+LL	0.000	-0.011	0.000	0.011	0.000	0.000	-0.000
	101:DL+0.6WL	0.001	-0.008	0.000	0.008	0.000	0.000	-0.000
	102:DL+0.6WL	-0.000	-0.008	0.000	0.008	0.000	0.000	-0.000
	103:DL+0.75LL	0.001	-0.011	0.000	0.011	0.000	0.000	-0.000
	104:DL+0.75LL	-0.000	-0.011	0.000	0.011	0.000	0.000	-0.000
	105:0.6DL+0.6'	0.000	-0.005	0.000	0.005	0.000	0.000	-0.000
	106:0.6DL+0.6'	-0.000	-0.006	0.000	0.006	0.000	0.000	-0.000
14	1:DL	-0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	2:DL EXTERN/	0.002	-0.002	0.000	0.003	0.000	0.000	0.000
	3:LL	0.001	-0.001	0.000	0.002	0.000	0.000	0.000
	4:WL NEG	0.001	0.000	0.000	0.001	0.000	0.000	-0.000
	5:WL B	-0.000	-0.001	0.000	0.001	0.000	0.000	0.000
	100:DL+LL	0.003	-0.003	0.000	0.005	0.000	0.000	0.000
	101:DL+0.6WL	0.003	-0.002	0.000	0.004	0.000	0.000	0.000
	102:DL+0.6WL	0.002	-0.003	0.000	0.003	0.000	0.000	0.000
	103:DL+0.75LL	0.004	-0.003	0.000	0.005	0.000	0.000	0.000
	104:DL+0.75LL	0.003	-0.004	0.000	0.005	0.000	0.000	0.000
	105:0.6DL+0.6'	0.002	-0.001	0.000	0.002	0.000	0.000	-0.000
	106:0.6DL+0.6'	0.001	-0.002	0.000	0.002	0.000	0.000	0.000
15	1:DL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	2:DL EXTERN/	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	3:LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	4:WL NEG	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	5:WL B	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	100:DL+LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	101:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	102:DL+0.6WL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	103:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	104:DL+0.75LL	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	105:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	-0.000
	106:0.6DL+0.6'	0.000	0.000	0.000	0.000	0.000	0.000	-0.000

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	**Env**	**FX** (kip)	**FY** (kip)	**FZ** (kip)	**MX** (kip`in)	**MY** (kip`in)	**MZ** (kip`in)
1	+ve	5.387	3.957	0.000	0.000	0.000	0.000
1	+ve	Load: 103	Load: 100	-	-	-	-
1	-ve	0.000	0.000	0.000	0.000	0.000	0.000
1	-ve	-	-	-	-	-	-
9	+ve	0.000	3.925	0.000	0.000	0.000	0.000
9	+ve	-	Load: 100	-	-	-	-
9	-ve	-5.468	0.000	0.000	0.000	0.000	0.000
9	-ve	Load: 103	-	-	-	-	-
10	+ve	6.339	9.832	0.000	0.000	0.000	0.000
10	+ve	Load: 100	Load: 100	-	-	-	-
10	-ve	0.000	0.000	0.000	0.000	0.000	0.000
10	-ve	-	-	-	-	-	-
15	+ve	0.000	9.886	0.000	0.000	0.000	0.000
15	+ve	-	Load: 100	-	-	-	-
15	-ve	-6.386	0.000	0.000	0.000	0.000	0.000
15	-ve	Load: 100	-	-	-	-	-

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	**L/C**	**FX** (kip)	**FY** (kip)	**FZ** (kip)	**MX** (kip`in)	**MY** (kip`in)	**MZ** (kip`in)
Max FX	10	100:DL+LL	**6.339**	9.832	0.000	0.000	0.000	0.000
Min FX	15	100:DL+LL	**-6.386**	9.886	0.000	0.000	0.000	0.000
Max FY	15	100:DL+LL	-6.386	**9.886**	0.000	0.000	0.000	0.000
Min FY	15	4:WL NEG	-0.997	**0.024**	0.000	0.000	0.000	0.000
Max FZ	1	1:DL	0.026	0.161	**0.000**	0.000	0.000	0.000
Min FZ	1	1:DL	0.026	0.161	**0.000**	0.000	0.000	0.000
Max MX	1	1:DL	0.026	0.161	0.000	**0.000**	0.000	0.000
Min MX	1	1:DL	0.026	0.161	0.000	**0.000**	0.000	0.000
Max MY	1	1:DL	0.026	0.161	0.000	0.000	**0.000**	0.000
Min MY	1	1:DL	0.026	0.161	0.000	0.000	**0.000**	0.000
Max MZ	1	1:DL	0.026	0.161	0.000	0.000	0.000	**0.000**
Min MZ	1	1:DL	0.026	0.161	0.000	0.000	0.000	**0.000**

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip¯in)	MY (kip¯in)	MZ (kip¯in)
1	1:DL	0.026	0.161	0.000	0.000	0.000	0.000
	2:DL EXTERN/	3.395	2.415	0.000	0.000	0.000	0.000
	3:LL	1.846	1.381	0.000	0.000	0.000	0.000
	4:WL NEG	1.293	0.630	0.000	0.000	0.000	0.000
	5:WL B	0.678	0.485	0.000	0.000	0.000	0.000
	100:DL+LL	5.266	3.957	0.000	0.000	0.000	0.000
	101:DL+0.6WL	4.196	2.954	0.000	0.000	0.000	0.000
	102:DL+0.6WL	3.827	2.867	0.000	0.000	0.000	0.000
	103:DL+0.75LL	5.387	3.896	0.000	0.000	0.000	0.000
	104:DL+0.75LL	5.110	3.830	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	2.828	1.924	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	2.459	1.837	0.000	0.000	0.000	0.000
9	1:DL	-0.024	0.160	0.000	0.000	0.000	0.000
	2:DL EXTERN/	-3.365	2.395	0.000	0.000	0.000	0.000
	3:LL	-1.830	1.370	0.000	0.000	0.000	0.000
	4:WL NEG	-1.569	0.732	0.000	0.000	0.000	0.000
	5:WL B	-0.101	0.263	0.000	0.000	0.000	0.000
	100:DL+LL	-5.219	3.925	0.000	0.000	0.000	0.000
	101:DL+0.6WL	-4.331	2.994	0.000	0.000	0.000	0.000
	102:DL+0.6WL	-3.449	2.712	0.000	0.000	0.000	0.000
	103:DL+0.75LL	-5.468	3.912	0.000	0.000	0.000	0.000
	104:DL+0.75LL	-4.807	3.701	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	-2.975	1.972	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	-2.094	1.690	0.000	0.000	0.000	0.000
10	1:DL	0.339	1.243	0.000	0.000	0.000	0.000
	2:DL EXTERN/	3.873	5.382	0.000	0.000	0.000	0.000
	3:LL	2.127	3.207	0.000	0.000	0.000	0.000
	4:WL NEG	1.013	0.388	0.000	0.000	0.000	0.000
	5:WL B	0.638	0.883	0.000	0.000	0.000	0.000
	100:DL+LL	6.339	9.832	0.000	0.000	0.000	0.000
	101:DL+0.6WL	4.820	6.857	0.000	0.000	0.000	0.000
	102:DL+0.6WL	4.595	7.155	0.000	0.000	0.000	0.000
	103:DL+0.75LL	6.263	9.204	0.000	0.000	0.000	0.000
	104:DL+0.75LL	6.094	9.427	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	3.135	4.207	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	2.910	4.505	0.000	0.000	0.000	0.000
15	1:DL	-0.340	1.244	0.000	0.000	0.000	0.000
	2:DL EXTERN/	-3.903	5.416	0.000	0.000	0.000	0.000
	3:LL	-2.143	3.225	0.000	0.000	0.000	0.000
	4:WL NEG	-0.997	0.024	0.000	0.000	0.000	0.000
	5:WL B	-0.697	1.639	0.000	0.000	0.000	0.000
	100:DL+LL	-6.386	9.886	0.000	0.000	0.000	0.000
	101:DL+0.6WL	-4.841	6.675	0.000	0.000	0.000	0.000
	102:DL+0.6WL	-4.662	7.644	0.000	0.000	0.000	0.000

Reactions Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	**L/C**	**FX** (kip)	**FY** (kip)	**FZ** (kip)	**MX** (kip˙in)	**MY** (kip˙in)	**MZ** (kip˙in)
	103:DL+0.75LL	-6.299	9.091	0.000	0.000	0.000	0.000
	104:DL+0.75LL	-6.164	9.817	0.000	0.000	0.000	0.000
	105:0.6DL+0.6'	-3.144	4.011	0.000	0.000	0.000	0.000
	106:0.6DL+0.6'	-2.964	4.980	0.000	0.000	0.000	0.000

TUBE TRUSS REACTIONS

SUPPORT REACTIONS -UNIT POUN FEET STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	100	5266.46	3957.39	0.00	0.00	0.00	0.00
	101	4196.40	2954.06	0.00	0.00	0.00	0.00
	102	3827.38	2867.17	0.00	0.00	0.00	0.00
	103	5386.99	3895.53	0.00	0.00	0.00	0.00
	104	5110.23	3830.36	0.00	0.00	0.00	0.00
	105	2828.28	1923.62	0.00	0.00	0.00	0.00
	106	2459.26	1836.73	0.00	0.00	0.00	0.00
9	100	-5219.20	3925.25	0.00	0.00	0.00	0.00
	101	-4330.71	2994.27	0.00	0.00	0.00	0.00
	102	-3449.39	2712.35	0.00	0.00	0.00	0.00
	103	-5467.92	3912.23	0.00	0.00	0.00	0.00
	104	-4806.92	3700.79	0.00	0.00	0.00	0.00
	105	-2975.10	1972.34	0.00	0.00	0.00	0.00
	106	-2093.78	1690.42	0.00	0.00	0.00	0.00
10	100	6339.13	9831.78	0.00	0.00	0.00	0.00
	101	4819.91	6857.30	0.00	0.00	0.00	0.00
	102	4594.79	7154.57	0.00	0.00	0.00	0.00
	103	6263.33	9204.42	0.00	0.00	0.00	0.00
	104	6094.49	9427.37	0.00	0.00	0.00	0.00
	105	3135.15	4207.38	0.00	0.00	0.00	0.00
	106	2910.03	4504.66	0.00	0.00	0.00	0.00
15	100	-6386.39	9886.05	0.00	0.00	0.00	0.00
	101	-4841.19	6675.24	0.00	0.00	0.00	0.00
	102	-4661.60	7644.17	0.00	0.00	0.00	0.00
	103	-6299.10	9090.64	0.00	0.00	0.00	0.00
	104	-6164.41	9817.33	0.00	0.00	0.00	0.00
	105	-3143.92	4010.98	0.00	0.00	0.00	0.00
	106	-2964.33	4979.90	0.00	0.00	0.00	0.00

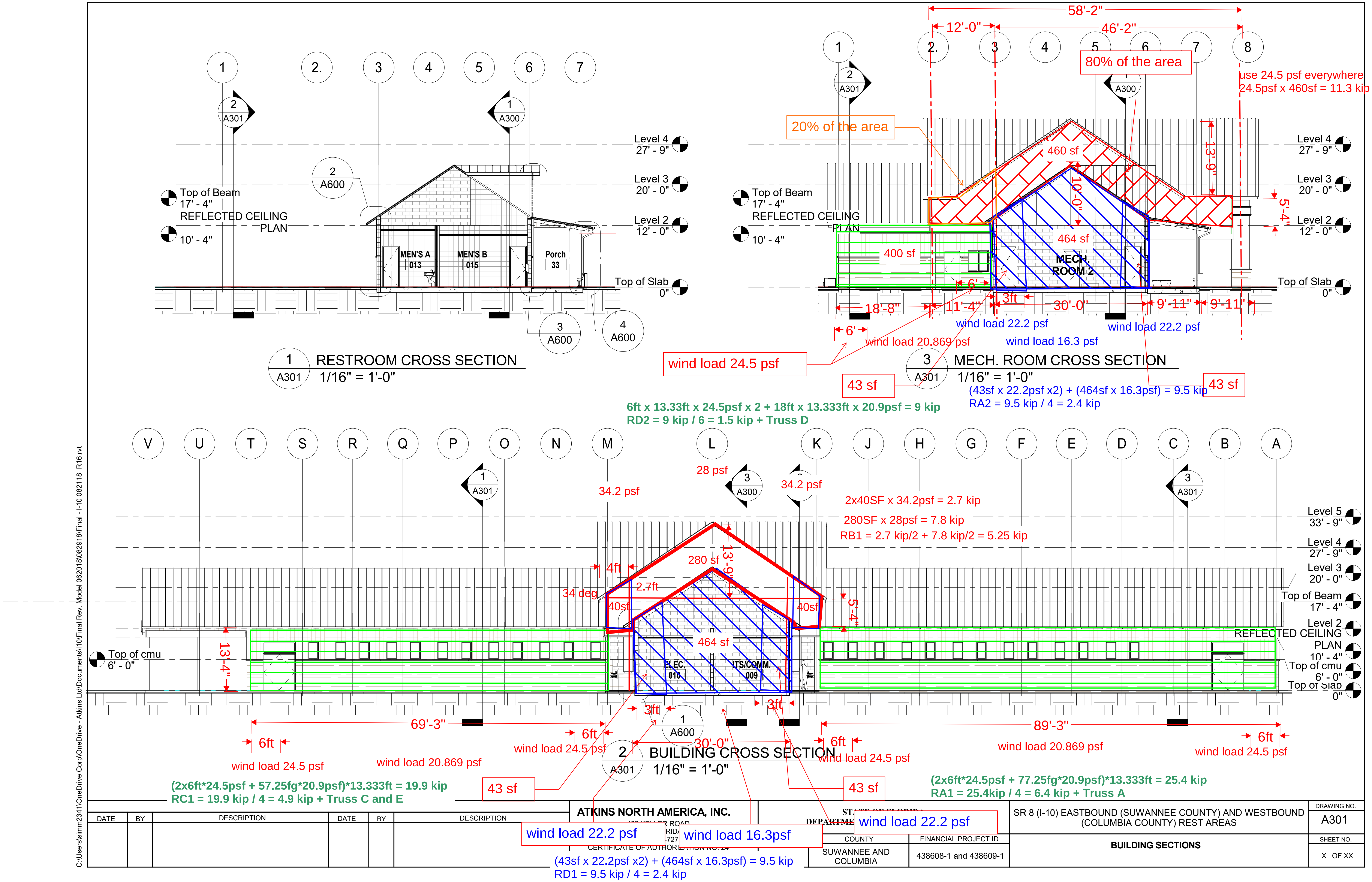
1. DL - SELFWEIGHT
2. DL - EXTERNAL
3. LL_r
4.WL - NEGATIVE
5.WL - POSITIVE

100. LC2 -DL+LL_r
101. LC5 -DL+0.6WL NEG
102. LC5 - DL+0.6WL POS
103. LC6a -DL+0.75LL_r+0.45WL NEG
104. LC6a -DL+0.75LL_r+0.45WL POS
106. LC7 -0.6DL+0,6WL NEG
107. LC7 -O.6DL+0.6WL POS

************** END OF LATEST ANALYSIS RESULT **************

133. FINISH

Rest Room Building Shear Walls



DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.

CERTIFICATE OF AUTHORIZATION NO. 24

STATE OF FLORIDA DEPARTMENT

COUNTY	FINANCIAL PROJECT ID
SUWANNEE AND COLUMBIA	438608-1 and 438609-1

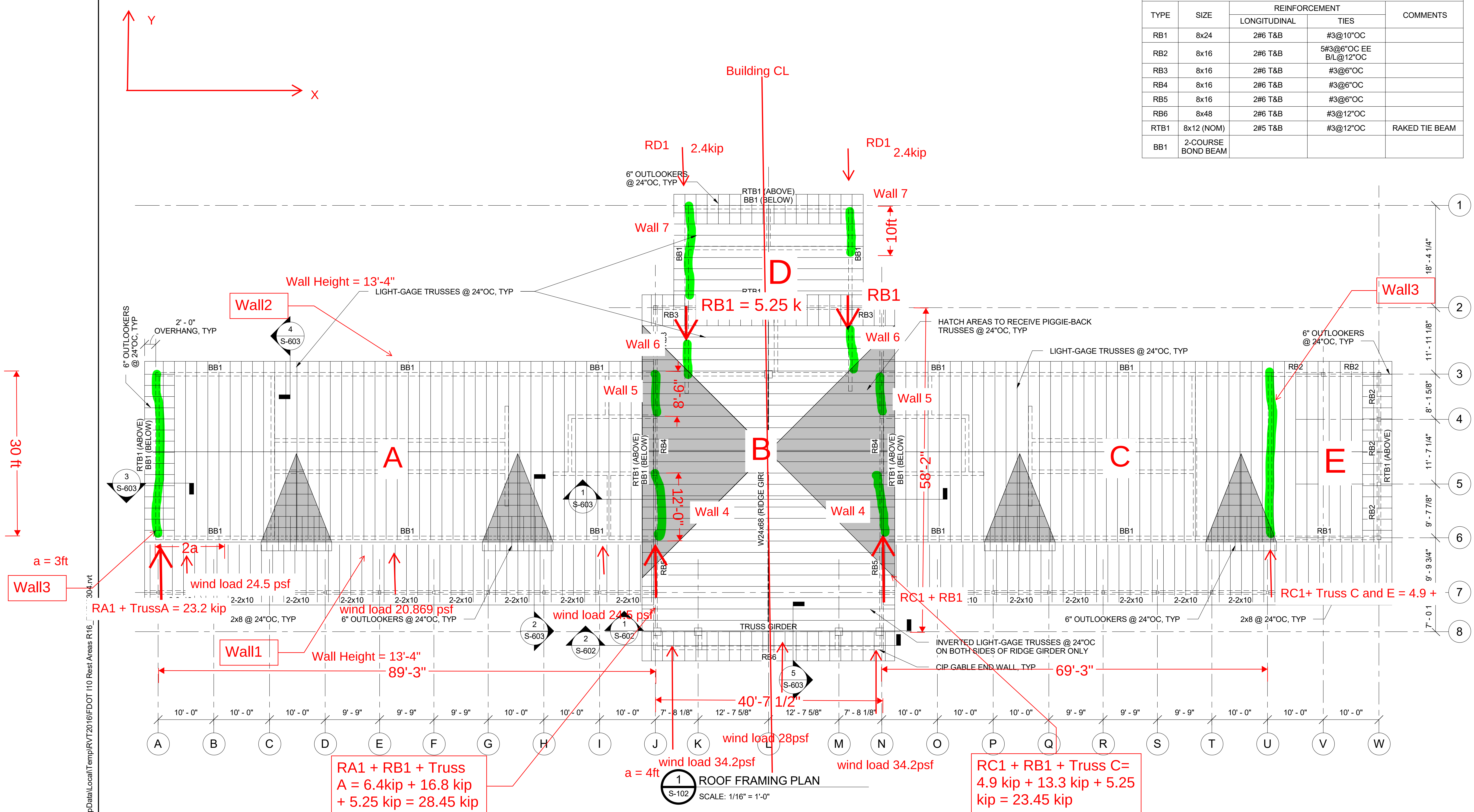
SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

BUILDING SECTIONS

DRAWING NO. A301

SHEET NO. X OF XX

BEAM SCHEDULE				
TYPE	SIZE	REINFORCEMENT		COMMENTS
		LONGITUDINAL	TIES	
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EE B/L@12"OC	
RB3	8x16	2#6 T&B	#3@6"OC	
RB4	8x16	2#6 T&B	#3@6"OC	
RB5	8x16	2#6 T&B	#3@6"OC	
RB6	8x48	2#6 T&B	#3@12"OC	
RTB1	8x12 (NOM)	2#5 T&B	#3@12"OC	RAKED TIE BEAM
BB1	2-COURSE BOND BEAM			



REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

ATKINS NORTH AMERICA, INC.
482 KELLER ROAD
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CERTIFICATE OF AUTHORIZATION NO. 24
DOUGLAS A. RAMIREZ , P.E. NO. 70993

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

ROOF FRAMING PLAN

DRAWING NO. S-102

SHEET NO. XX OF XX

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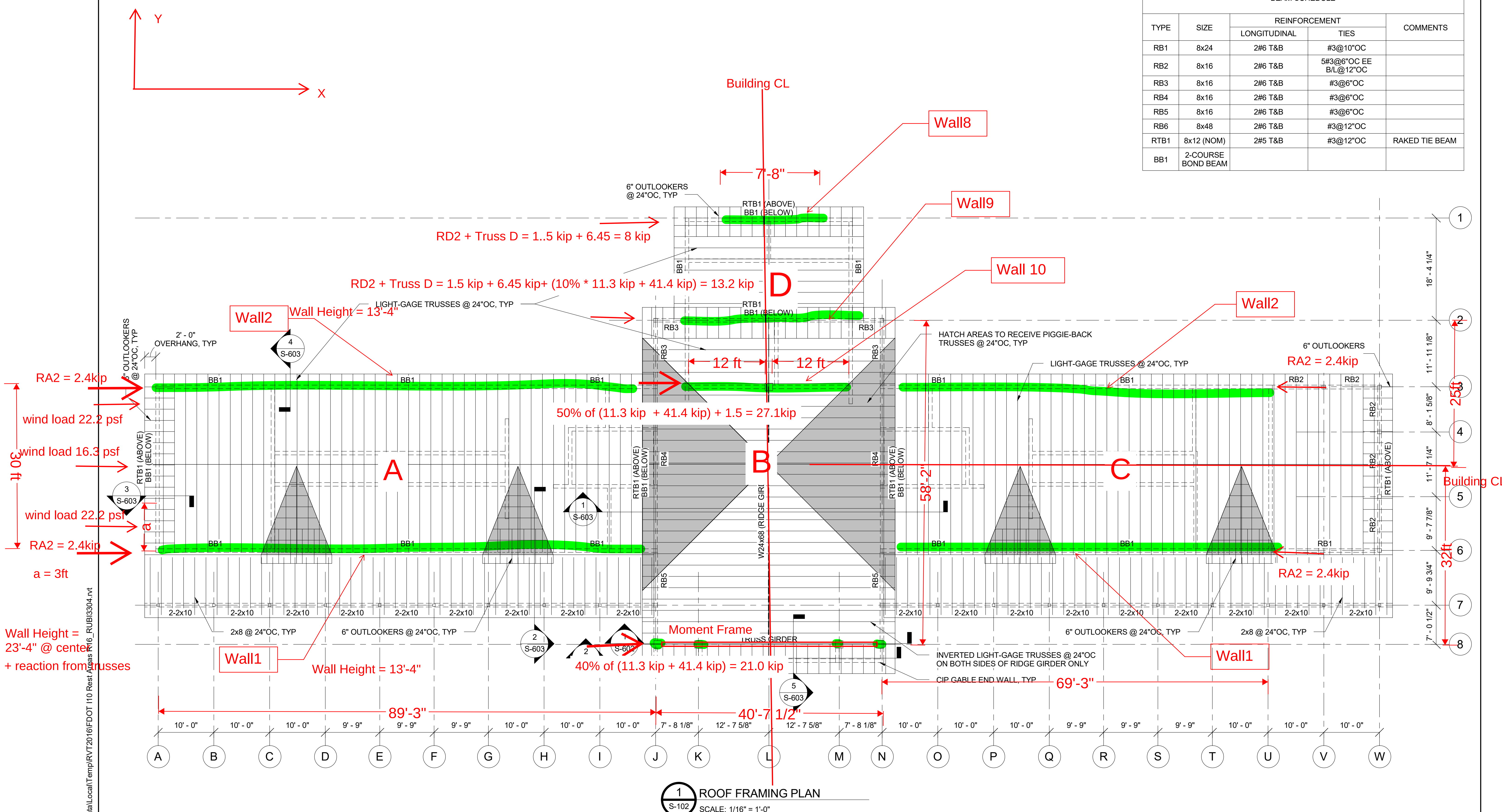
BEAM SCHEDULE				
TYPE	SIZE	REINFORCEMENT		COMMENTS
		LONGITUDINAL	TIES	
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EE B/L@12"OC	
RB3	8x16	2#6 T&B	#3@6"OC	
RB4	8x16	2#6 T&B	#3@6"OC	
RB5	8x16	2#6 T&B	#3@6"OC	
RB6	8x48	2#6 T&B	#3@12"OC	
RTB1	8x12 (NOM)	2#5 T&B	#3@12"OC	RAKED TIE BEAM
BB1	2-COURSE BOND BEAM			



1 S-102 ROOF FRAMING PLAN
SCALE: 1/16" = 1'-0"

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REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

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STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

ROOF FRAMING PLAN

DRAWING NO. S-102
SHEET NO. XX OF XX

SUBJECT: I-10 Rest Area Lateral Analysis
BY: ALM DATE: 5/13/2020
CHCK BY: NJA DATE: 7/6/2020

Masonry Wall Design
PROJ. # B190988.00

Purpose: Determine loads on Masonry Shear walls for TEDDS input.

Determine wind loads on shear walls. See attached figures

$H_{wall} := 13.333ft$ Wall height wall 1 and 2

$H_{wall3} := 23.333ft$ Height of wall 3 at the peak

$L_{wall1} := 89.25ft$

$L_{wall2} := 89.25ft$

$L_{wall3} := 30ft$

$$Area_{wall1} := H_{wall} \cdot L_{wall1} = 1189.97 \cdot ft^2$$

$$Area_{wall2} := H_{wall} \cdot L_{wall2} = 1189.97 \cdot ft^2$$

$$Area_{wall3} := L_{wall3} \cdot H_{wall} + 2 \cdot \left[\frac{L_{wall3}}{2} \cdot .5 \cdot \left(H_{wall3} - H_{wall}\right)\right] = 549.99 \cdot ft^2$$

$a := 3ft$

Each end wall segment is 10% of the total wall area

Wind loads from Design Basis

$Wind_{end} := 24.5psf$

$Wind_{mid} := 20.89psf$

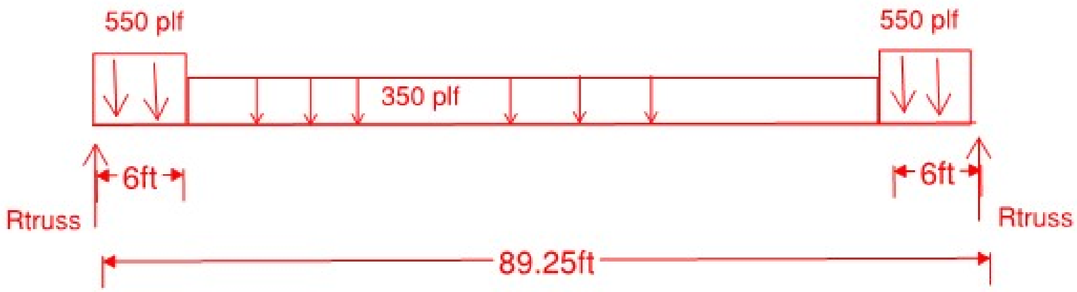
$Wind_{sideend} := 22.2psf$

$Wind_{sidemid} := 16.3psf$

$$Ra_1 := \frac{2 \cdot \left[Wind_{end} \cdot 2 \cdot a \cdot \left(\frac{H_{wall}}{2}\right)\right] + Wind_{mid} \cdot (89.25ft - 4 \cdot a) \cdot \frac{H_{wall}}{2}}{2} = 6.359 \cdot kip$$

plus the load from the roof trusses

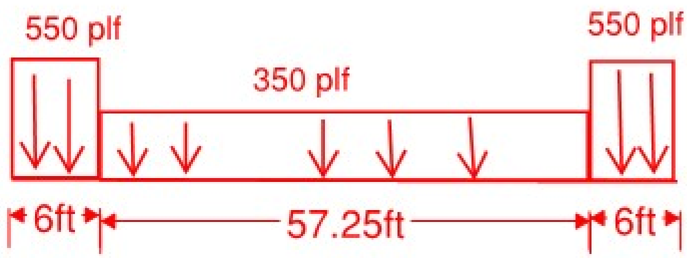
Wall 3 also includes the shear from the Restroom truss reactions. See Risa file.



loads into diaphragm from truss A reactions

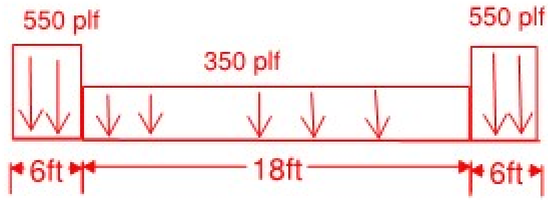
$$R_{trussA} := \frac{350plf \cdot (L_{wall1} - 12ft) + 550plf \cdot 12ft}{2} = 16.819 \cdot kip$$

$$Ra_{1Total} := Ra_1 + R_{trussA} = 23.178 \cdot kip$$



loads into diaphragm from truss C reactions. reactions are the same as Truss A but different spans.

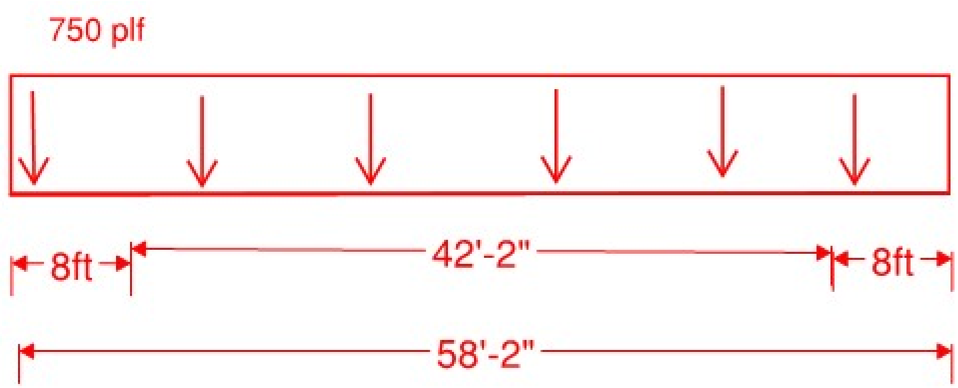
$$R_{trussD} := \frac{350plf \cdot 57.25ft + (2 \cdot 550plf \cdot 6ft)}{2} = 13.319 \cdot kip$$



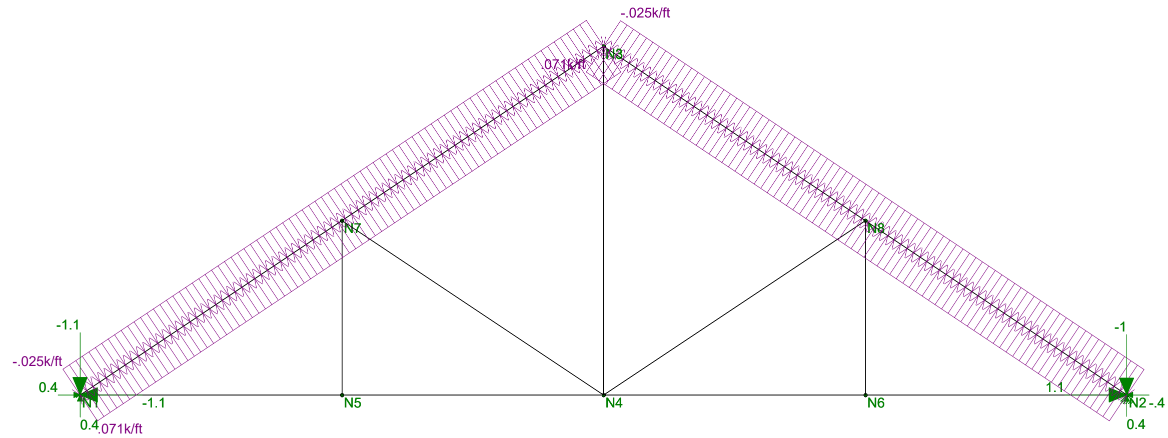
Loads into diaphragm from Truss D reactions. Reactions are the same as Truss A but different spans

$$R_{trussD} := \frac{350\text{plf} \cdot 18\text{ft} + 550\text{plf} \cdot 12\text{ft}}{2} = 6.45 \cdot \text{kip}$$

Truss reactions from Truss B. See STADD File.



$$R_{trussB} := \frac{750\text{plf} \cdot 58.167\text{ft}}{2} = 21.813 \cdot \text{kip}$$



Loads: LC 8, Wind 1
Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

	Bathroom Roof Truss	SK - 1
ALM		July 7, 2020 at 8:57 AM
B190988.00		Bathroom Truss Reactions.r3d

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	Y	-.04	-.04	0	0
2	M2	Y	-.04	-.04	0	0

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	Y	-.04	-.04	0	0
2	M2	Y	-.04	-.04	0	0

Member Distributed Loads (BLC 5 : Wind 1 - Truss 3A)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	y	-.025	-.025	0	0
2	M2	y	-.025	-.025	0	0

Member Distributed Loads (BLC 6 : Wind 2 - Truss 3A)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	y	.071	.071	0	0
2	M2	y	.071	.071	0	0

Member Distributed Loads (BLC 7 : Wind 3 - Truss 3)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	y	-.022	-.022	0	0
2	M2	y	-.022	-.022	0	0

Member Distributed Loads (BLC 8 : Wind 4 - Truss 3)

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Lo...	End Location[ft,%]
1	M1	y	.049	.049	0	0
2	M2	y	.049	.049	0	0

Basic Load Cases

	BLC Description	Category	X Gr...	Y Gra...	Z Gra...	Joint	Point	Distrib...	Area(Member)	Surface(Plate/Wall)
1	DL	None						2		
2	LL	None						2		
3	SW	None		-1						
5	Wind 1 - Truss 3A	None						2		
6	Wind 2 - Truss 3A	None						2		
7	Wind 3 - Truss 3	None						2		
8	Wind 4 - Truss 3	None						2		

Load Combinations

	Description	S...	P...	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	BLC	Fa...
1	DL+LL				1	1	2	1	3	1												
2	DL+0.75LL+0.75*0.6WL1				1	1	3	1	2	.75	5	.45										
3	0.6DL+0.6WL1				1	.6	3	.6	5	.6												
4	DL+0.75LL+0.75*0.6WL2				1	1	3	1	2	.75	6	.45										
5	0.6DL+0.6WL2				1	.6	3	.6	6													

Load Combinations (Continued)

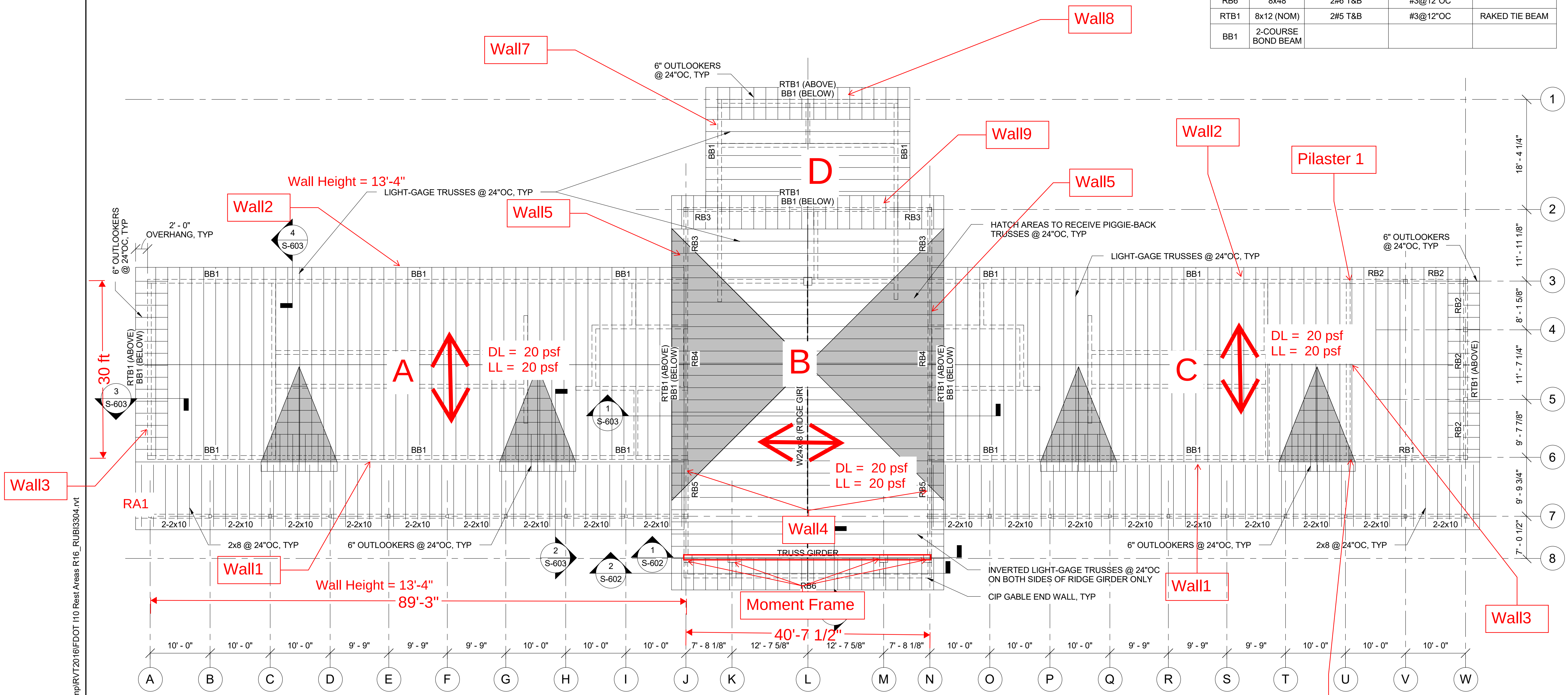
	Description	S...	P...	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	BLC	Fa...
6	DL+0.75LL+0.75*0.6WL3				1	1	3	1	2	.75	7	.45												
7	0.6DL+0.6WL3				1	.6	3	.6	7															
8	Wind 1	Yes			5	1																		
9	Wind 2	Yes			6	1																		
10	Wind 3	Yes			7	1																		
11	Wind 4	Yes			8	1																		

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	1.078	9	.371	8	0	11	0	11	0	11	0	11
2		min	-.384	8	-1.048	9	0	8	0	8	0	8	0	8
3	N1	max	.38	8	.385	8	0	11	0	11	0	11	.18	8
4		min	-1.078	9	-1.082	9	0	8	0	8	0	8	-.503	9
5	Totals:	max	0	11	.756	8	0	11						
6		min	-.004	8	-2.13	9	0	8						

BEAM SCHEDULE

TYPE	SIZE	REINFORCEMENT: LONGITUDINAL	REINFORCEMENT: TIES	COMMENTS
RB1	8x24	2#6 T&B	#3@10"OC	
RB2	8x16	2#6 T&B	5#3@6"OC EE B/L@12"OC	
RB3	8x16	2#6 T&B	#3@6"OC	
RB4	8x16	2#6 T&B	#3@6"OC	
RB5	8x16	2#6 T&B	#3@6"OC	
RB6	8x48	2#6 T&B	#3@12"OC	
RTB1	8x12 (NOM)	2#5 T&B	#3@12"OC	RAKED TIE BEAM
BB1	2-COURSE BOND BEAM			



1 S-102 ROOF FRAMING PLAN

SCALE: 1/16" = 1'-0"

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
		REVISIONS			

ATKINS NORTH AMERICA, INC.
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STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	SUWANNEE COLUMBIA	438608-1-52-01 438609-1-52-01

SR 8 (I-10) EASTBOUND (SUWANNEE COUNTY) AND WESTBOUND (COLUMBIA COUNTY) REST AREAS

ROOF FRAMING PLAN

DRAWING NO. S-102

SHEET NO. XX OF XX

SUBJECT:_I-10 Rest Area Masonry Wall Design
PROJ. #_B190988.00_

BY:_ALM_ DATE: 5/13/2020 SHEET NO. 1 of 1
CHCK BY:_ DATE: _

Purpose: Determine the Gravity Loads at Masonry load bearing walls from Roof for input into TEDDS.

$$Wall1 := \frac{30ft}{2} + 2ft + \frac{11.75ft}{2} = 22.875 \cdot ft$$ Tributary roof area of wall 1 includes half the patio roof

$$Wall2 := \frac{30ft}{2} + 2ft = 17 \cdot ft$$

$$Wall3 := 2ft + 1ft = 3 \cdot ft$$

$$Wall4 := \frac{40.625ft}{2} + 2ft = 22.313 \cdot ft$$

Roof Loads

$$DL := 20psf$$

$$SW_{truss} := 10psf$$ assumed

$$DL_{piggie} := 10psf$$ assumed

$$LL := 20psf$$

Gravity loads along the top of the wall:

$$DL_{Wall1} := (DL + SW_{truss}) \cdot Wall1 = 686.25 \cdot plf$$

$$LL_{Wall1} := LL \cdot Wall1 = 457.5 \cdot plf$$

$$DL_{Wall2} := (DL + SW_{truss}) \cdot Wall2 = 510 \cdot plf$$

$$LL_{Wall2} := LL \cdot Wall2 = 340 \cdot plf$$

$$DL_{Wall3} := (DL + SW_{truss}) \cdot Wall3 = 90 \cdot plf$$

$$LL_{Wall3} := LL \cdot Wall3 = 60 \cdot plf$$

$$DL_{Wall4} := (DL + SW_{truss} + DL_{piggie}) \cdot Wall4 = 892.5 \cdot plf$$

$$LL_{Wall4} := LL \cdot Wall4 = 446.25 \cdot plf$$

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

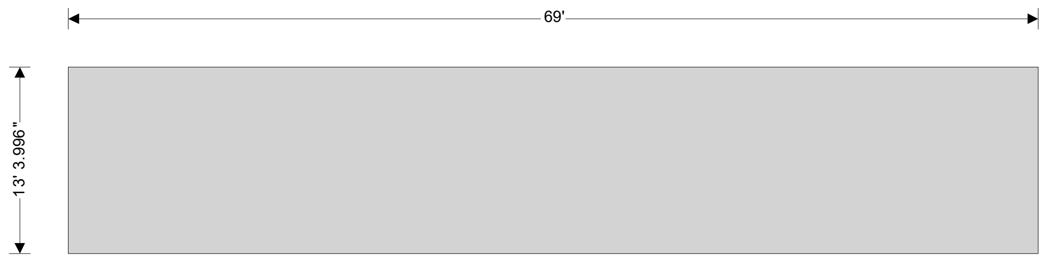
Masonry wall panel details

Bathroom Masonry Walls Wall 1 mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **69** ft

Panel height h = **13.333** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

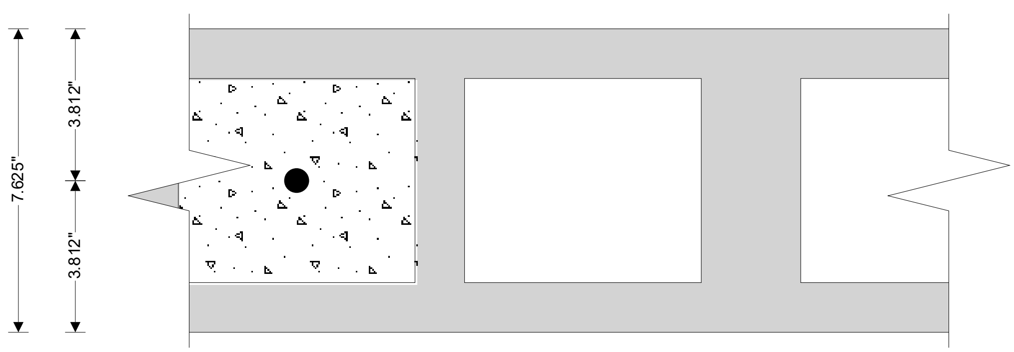
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **2800** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

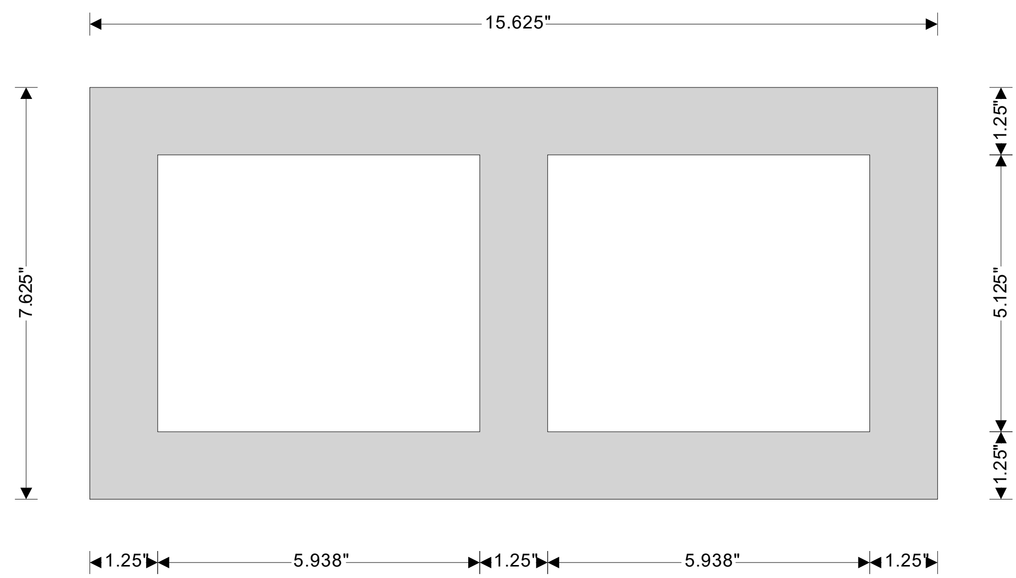
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in²/ft

Area of grout $A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in²/ft

Density of grout γ_{grout} = **140** lb/ft³

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **2250** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2025000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **810000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **38** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 48 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **24.5** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e = \mathbf{0.4}$

Seismic load from wall $E_{wall} = max(F_p, 0.1) \times w_{SW} = \mathbf{17.4}$ psf

Additional seimic load $E_{add} = \mathbf{0}$ psf

Seismic lateral load on panel $E = E_{wall} + E_{add} = \mathbf{17.4}$ psf

Lateral in-plane loads

Wind shear load on wall $V_W = \mathbf{6600}$ lbs

Vertical loading details

Dead load at supported level DL = **687** lb/ft

Live roof load at supported level $LL_r = \mathbf{510}$ lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS} = \mathbf{0.2}$

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.053)

Load combination no.2 DL + LL (0.053)

Load combination no.5 $DL + 0.6 \times W$ (0.325)

Load combination no.7 $DL + 0.75 \times LL + 0.45 \times W + 0.75 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.220)

Load combination no.8 $DL + 0.75 \times LL + 0.525 \times E_h + 0.525 \times E_v + 0.75 \times SL$ (0.196)

Load combination no.9 $0.6 \times DL + 0.6 \times W$ (0.365)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = \mathbf{52.7}$ in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = \mathbf{358.4}$ in⁴/ft

Section modulus $S = I / c = \mathbf{94}$ in³/ft

Radius of gyration $r = \sqrt{[I / A]} = \mathbf{2.608}$ in

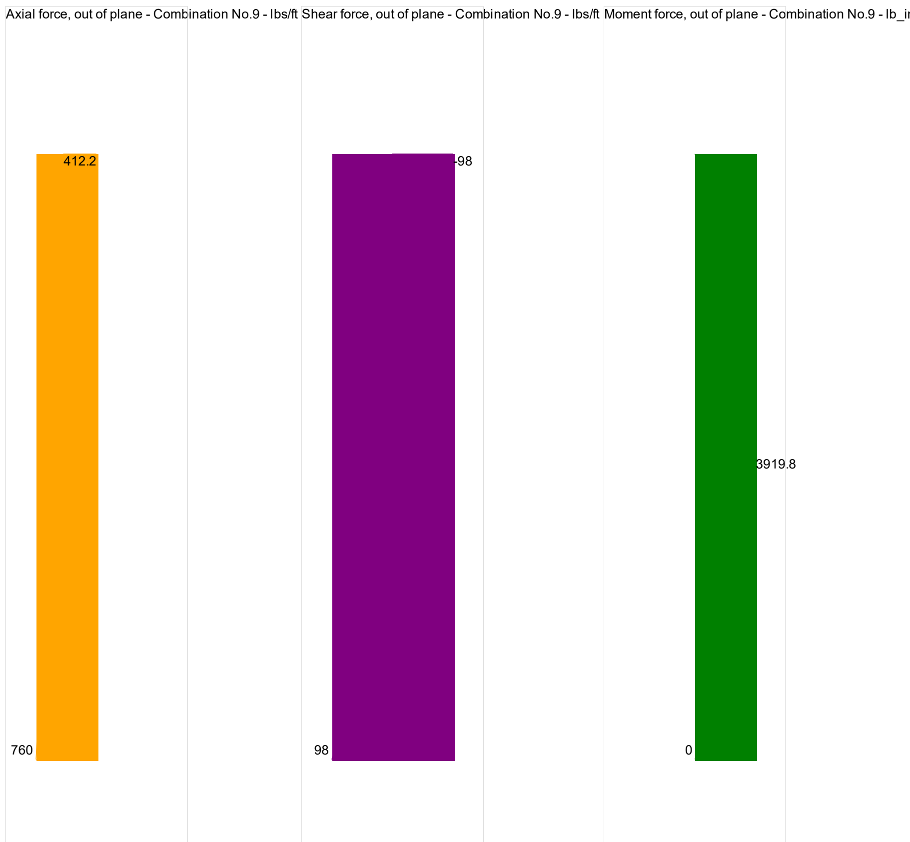
Effective height factor $K = \mathbf{1}$

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell9}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell15}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell21}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell22}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell23}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell25}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell26}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell27}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell28}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell29}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell31}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell32}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell33}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell34}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell35}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell37}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell38}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell39}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell40}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell41}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell43}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell44}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell45}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell46}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell47}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell49}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell50}^2) = \mathbf{222107378}$ in⁴

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) =$ **536491** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 6.67 ft

Axial load at max moment loc. of panel $P =$ **586** lb/ft

Compressive stress due to axial load $f_a = P / A =$ **11.1** psi

Slenderness ratio $(K \times h) / r =$ **61.355** < 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **454.5** psi

$f_a / F_a =$ **0.024**

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **23918** lb/ft

$P / P_a =$ **0.025**

Bending moment at max. moment loc. of panel $M =$ **3920** lb_in/ft

Depth of reinforcement $d =$ **3.813** in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) =$ **48** in

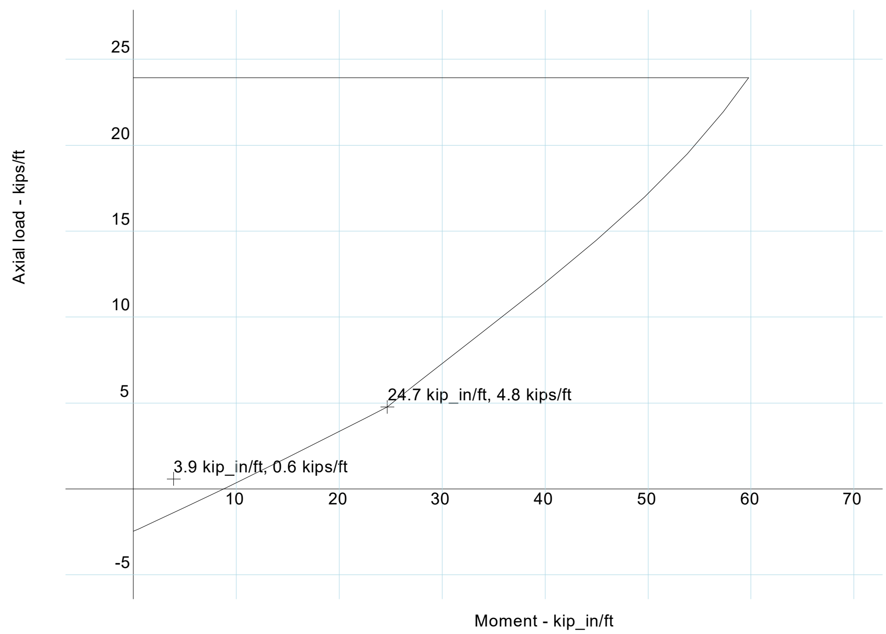
Modular ratio $n = E_s / E_m =$ **14.321**

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m =$ **1013** psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal})$ = **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m$ = **1012.5** psi
Tension at balance point	$T_{bal} = A_s \times F_s$ = **2454** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2$ = **7222** lb/ft
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal}$ = **4768** lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3)$ = **24673** lb_in/ft
Maximum moment from interaction diagram	M_c = **10742** lb_in/ft
	M / M_c = **0.365**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force	V = **98** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **7.4** in^2/ft
Shear stress	$f_v = V / A_{nv}$ = **13.2** psi
Compressive force	$N_v = 0.60 \times P_{DL,t,out} / 1\ ft$ = **412.2** lb/ft
Moment	M = **0** lb_in/ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\ psi)} + 0.25 \times N_v / A$ =**96.8** psi
Allowable shear stress	$F_v = \min(F_{vm}, 3 \times \sqrt{(f'_m \times 1\ psi)})$ = **96.8** psi
Shear utilization	f_v / F_v = **0.136**

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

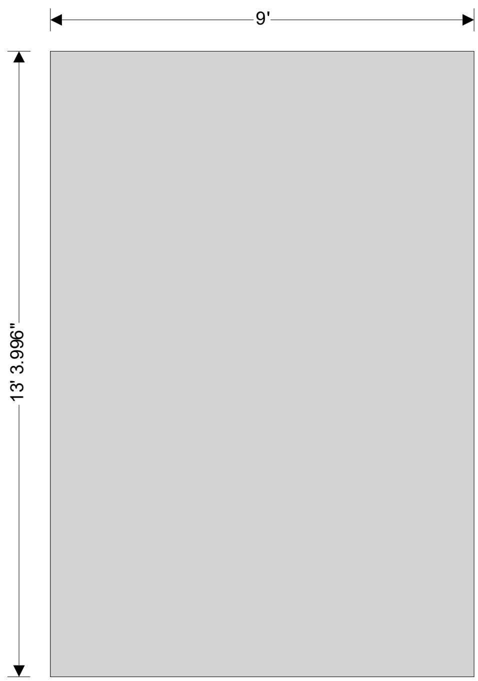
Masonry wall panel details

Bathroom Masonry Walls Wall 1 mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **9** ft

Panel height h = **13.333** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

Seismic wall classification Nonparticipating

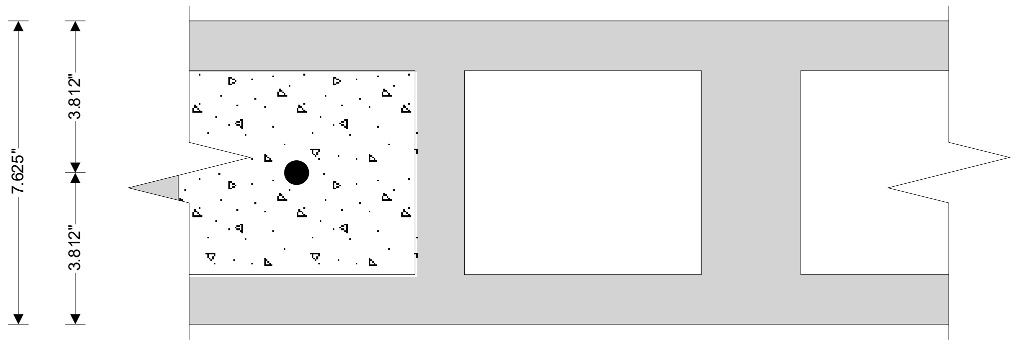
No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in

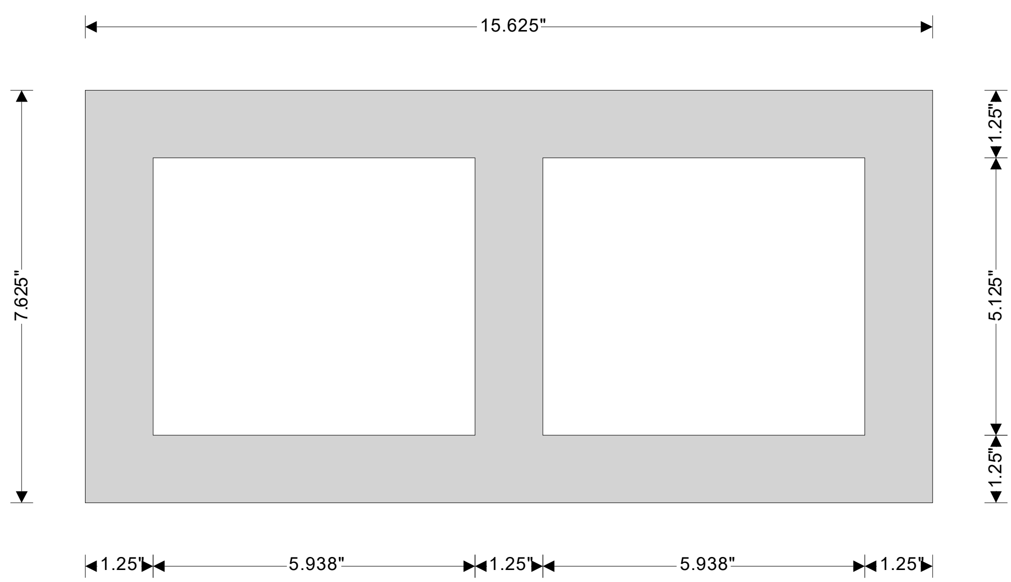
Tekla Tedds	Project I-10 Rest Area			Job Ref. B190988.00	
	Section Bathroom Wall 1, split			Sheet no./rev. 2	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **2800** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **2250** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2025000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **810000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **38** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 48 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **24.5** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **17.4** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **17.4** psf

Lateral in-plane loads

Wind shear load on wall V_W = **1100** lbs

Vertical loading details

Dead load at supported level DL = **687** lb/ft

Live roof load at supported level LL_r = **510** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.053)

Load combination no.2 DL + LL (0.053)

Load combination no.5 $DL + 0.6 \times W$ (0.325)

Load combination no.7 $DL + 0.75 \times LL + 0.45 \times W + 0.75 \times (LL_r$ or SL or RL) (0.220)

Load combination no.8 $DL + 0.75 \times LL + 0.525 \times E_h + 0.525 \times E_v + 0.75 \times SL$ (0.196)

Load combination no.9 $0.6 \times DL + 0.6 \times W$ (0.365)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **52.7** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **358.4** in⁴/ft

Section modulus $S = I / c$ = **94** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.608** in

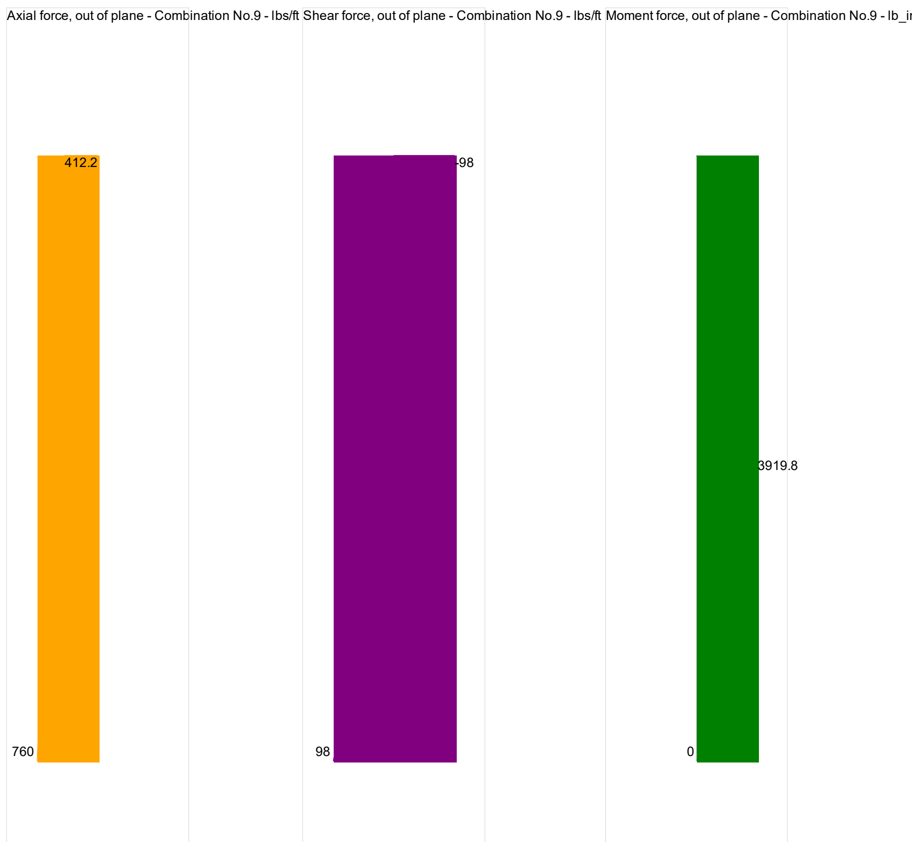
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia — $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) =$ **585323** in^4

Net section modulus — $S_{x\_net} = I_{x\_net} / (L / 2) =$ **10839** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location — 6.67 ft

Axial load at max moment loc. of panel — $P =$ **586** lb/ft

Compressive stress due to axial load — $f_a = P / A =$ **11.1** psi

Slenderness ratio — $(K \times h) / r =$ **61.355** < 99

Allowable compressive stress due to axial load — $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **454.5** psi

$f_a / F_a =$ **0.024**

Allowable compressive force — $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **23918** lb/ft

$P / P_a =$ **0.025**

Bending moment at max. moment loc. of panel — $M =$ **3920** lb_in/ft

Depth of reinforcement — $d =$ **3.813** in

Effective width per bar — $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) =$ **48** in

Modular ratio — $n = E_s / E_m =$ **14.321**

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m =$ **1013** psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m =$ **1012.5** psi

Tension at balance point $T_{bal} = A_s \times F_s =$ **2454** lb/ft

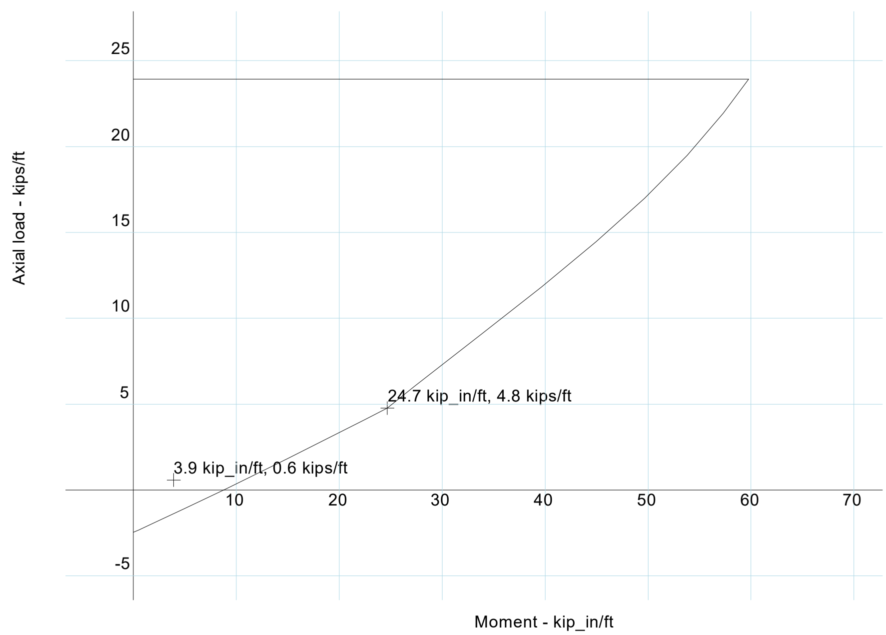
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **7222** lb/ft

Axial load at balance point $P_{bal} = C_{bal} – T_{bal} =$ **4768** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **24673** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **10742** lb_in/ft

$M / M_c =$ **0.365**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force $V =$ **98** lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) =$ **7.4** in²/ft

Shear stress $f_v = V / A_{nv} =$ **13.2** psi

Compressive force $N_v = 0.60 \times P_{DL,t,out} / 1\text{ ft} =$ **412.2** lb/ft

Moment $M =$ **0** lb_in/ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_v / A$ =**96.8** psi

Allowable shear stress $F_v = \min(F_{vm},3 \times \sqrt{(f'_m \times 1\text{ psi})}) =$ **96.8** psi

Shear utilization $f_v / F_v =$ **0.136**

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

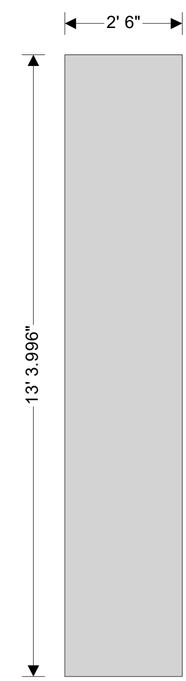
Masonry wall panel details

Bathroom Masonry Walls Wall 1 between windows - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **2.5** ft

Panel height h = **13.333** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

Seismic wall classification Nonparticipating

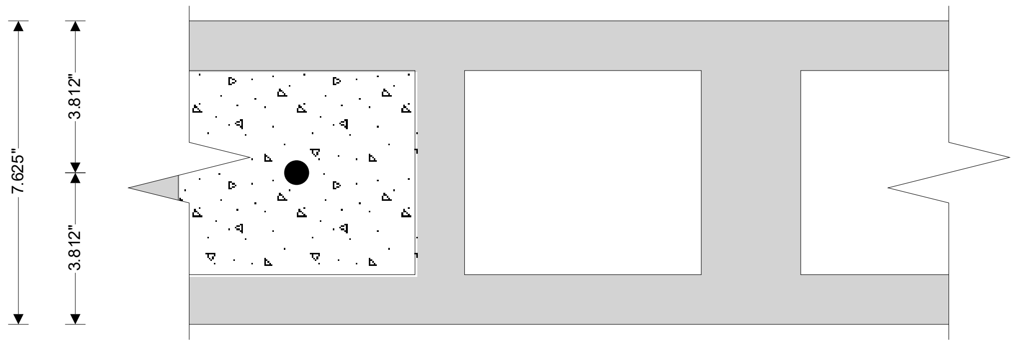
No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Bathroom Wall 1, between windows				Sheet no./rev. 2	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **2800** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

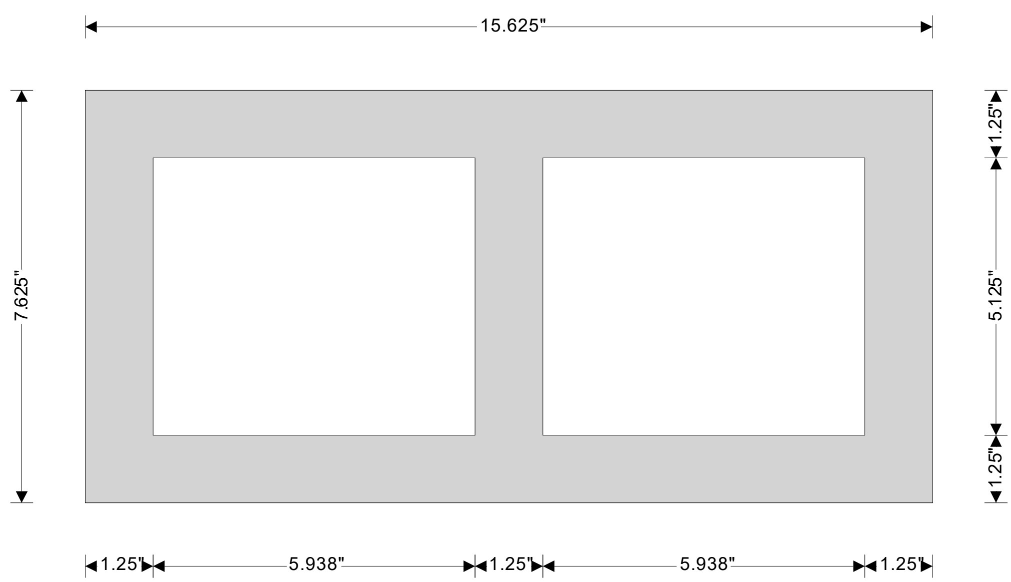
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **2250** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2025000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **810000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **38** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 48 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **49** psf

Wind load on parapet W_p = **24.5** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **17.4** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **17.4** psf

Lateral in-plane loads

Wind shear load on wall V_W = **550** lbs

Vertical loading details

Dead load at supported level DL = **1360** lb/ft

Live roof load at supported level LL_r = **1020** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.081)

Load combination no.2 DL + LL (0.081)

Load combination no.5 DL + 0.6 × W (0.546)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.348)

Load combination no.8 DL + 0.75 × LL + 0.525 × E_h + 0.525 × E_v + 0.75 × SL (0.163)

Load combination no.9 0.6 × DL + 0.6 × W (0.647)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **52.7** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **358.4** in⁴/ft

Section modulus $S = I / c$ = **94** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.608** in

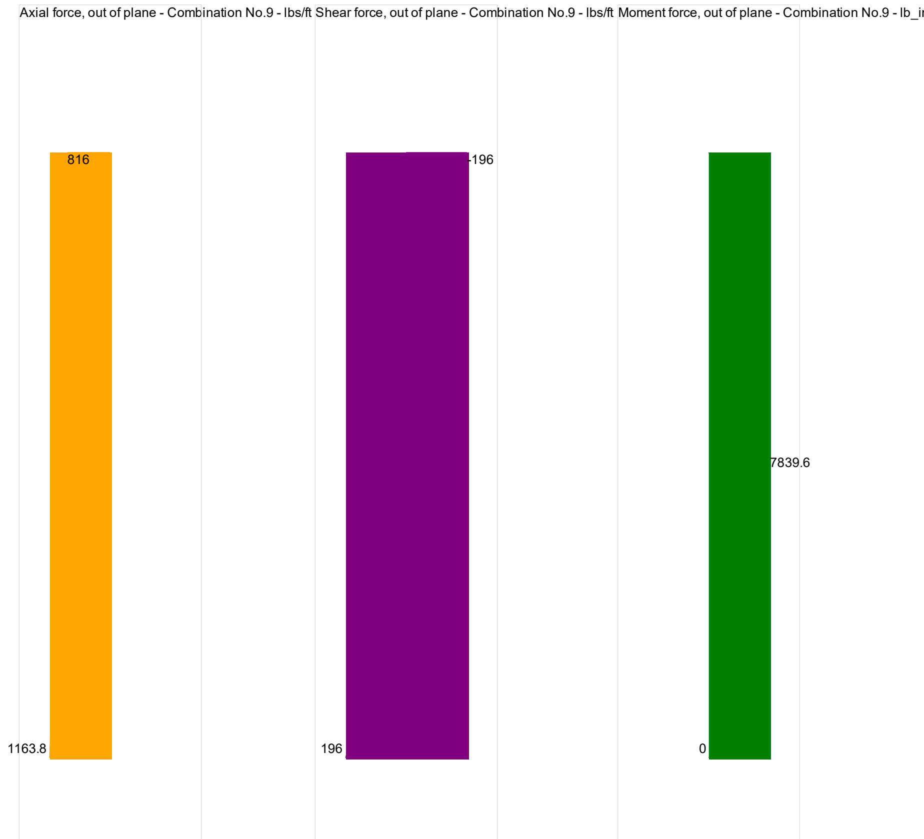
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 =$ **17156** in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) =$ **1144** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 6.67 ft

Axial load at max moment loc. of panel P = **990** lb/ft

Compressive stress due to axial load $f_a = P / A =$ **18.8** psi

Slenderness ratio $(K \times h) / r =$ **61.355** < 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **454.5** psi

$f_a / F_a =$ **0.041**

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **23918** lb/ft

$P / P_a =$ **0.041**

Bending moment at max. moment loc. of panel M = **7840** lb_in/ft

Depth of reinforcement d = **3.813** in

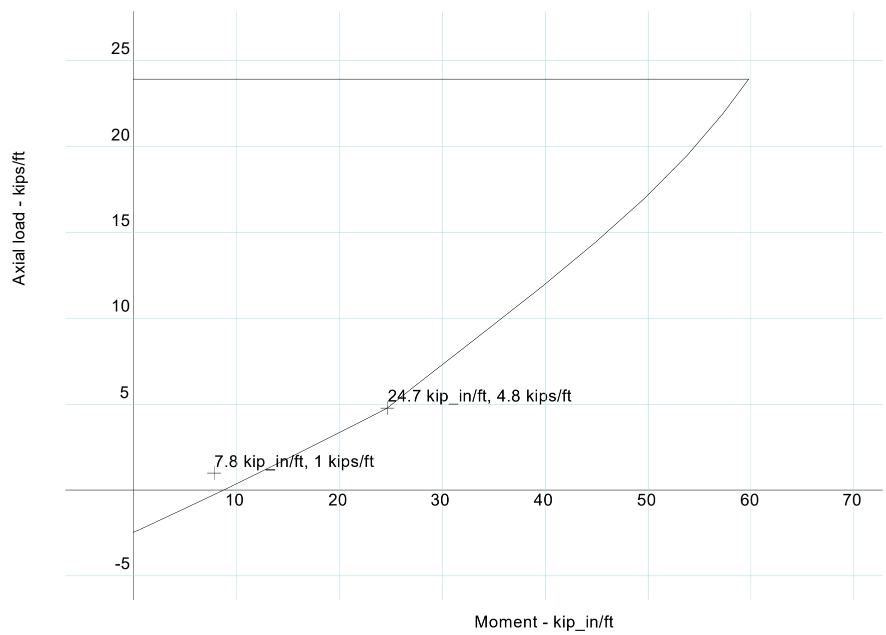
Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) =$ **48** in

Modular ratio $n = E_s / E_m =$ **14.321**

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m =$ **1013** psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s =$ **0.001103**
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m =$ **1012.5** psi
Tension at balance point	$T_{bal} = A_s \times F_s =$ **2454** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **7222** lb/ft
Axial load at balance point	$P_{bal} = C_{bal} – T_{bal} =$ **4768** lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **24673** lb_in/ft
Maximum moment from interaction diagram	$M_c =$ **12115** lb_in/ft
	$M / M_c =$ **0.647**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force	$V =$ **196** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) =$ **7.4** in^2/ft
Shear stress	$f_v = V / A_{nv} =$ **26.3** psi
Compressive force	$N_v = 0.60 \times P_{DL,t,out} / 1$ ft $=$ **816** lb/ft
Moment	$M =$ **0** lb_in/ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_v / A$ =**98.7** psi
Allowable shear stress	$F_v = \min(F_{vm},3 \times \sqrt{(f'_m \times 1\text{ psi})}) =$ **98.7** psi
Shear utilization	$f_v / F_v =$ **0.267**

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Bathroom Wall 2				Sheet no./rev. 1	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

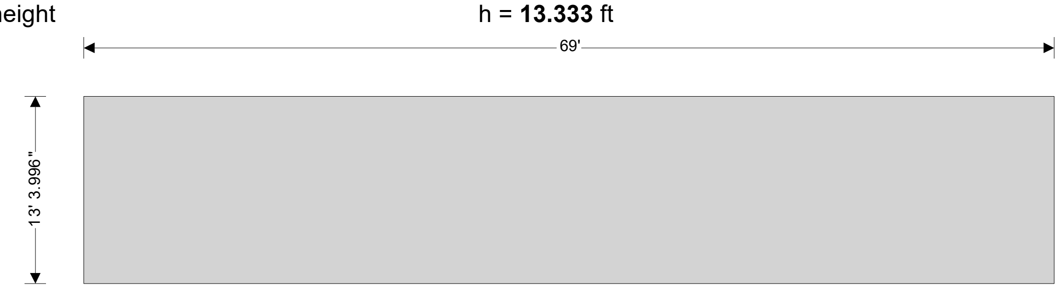
Masonry wall panel details

Bathroom Masonry Walls Wall 1 mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **69** ft

Panel height h = **13.333** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

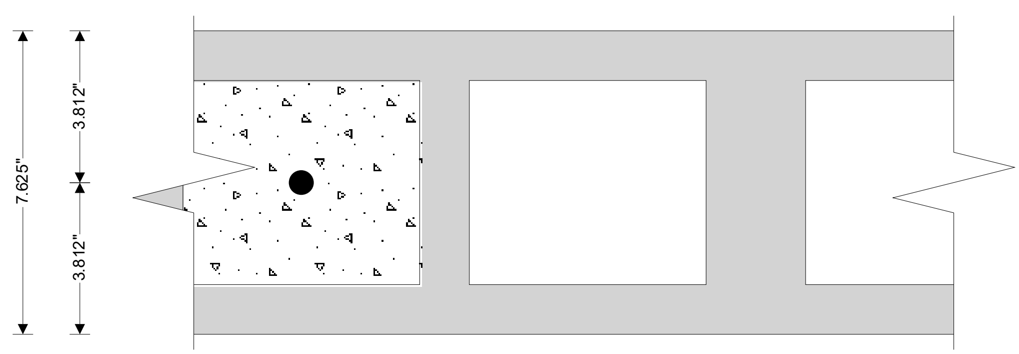
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **2800** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Tekla Tedds	Project I-10 Rest Area			Job Ref. B190988.00	
	Section Bathroom Wall 2			Sheet no./rev. 2	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

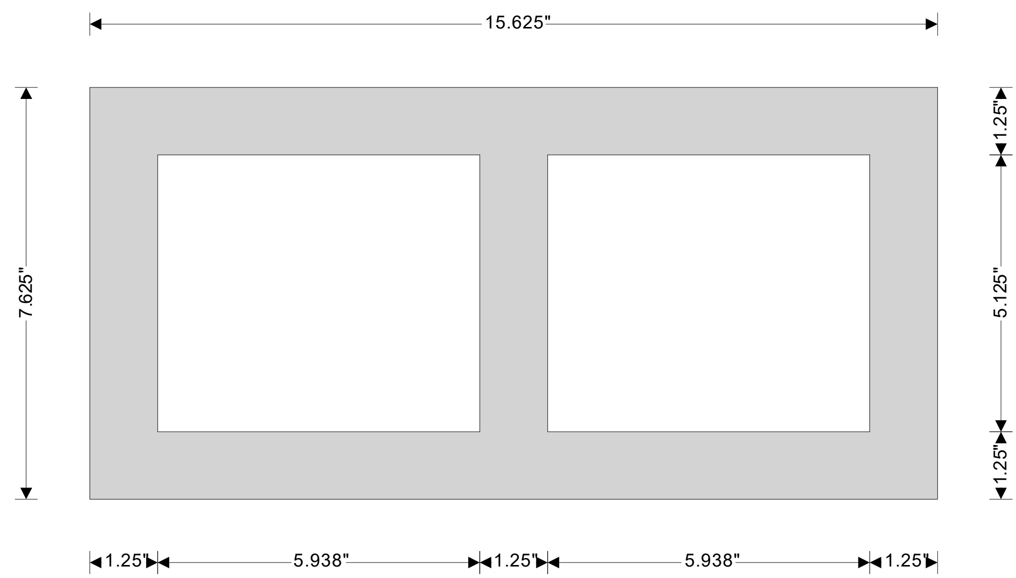
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in²/ft

Area of grout $A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in²/ft

Density of grout γ_{grout} = **140** lb/ft³

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **2250** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2025000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **810000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **38** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 48 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **24.5** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e =$ **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW} =$ **17.4** psf

Additional seimic load $E_{add} =$ **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add} =$ **17.4** psf

Lateral in-plane loads

Wind shear load on wall $V_W =$ **6600** lbs

Vertical loading details

Dead load at supported level DL = **687** lb/ft

Live roof load at supported level $LL_r =$ **510** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS} =$ **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.053)

Load combination no.2 DL + LL (0.053)

Load combination no.5 $DL + 0.6 \times W$ (0.325)

Load combination no.7 $DL + 0.75 \times LL + 0.45 \times W + 0.75 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.220)

Load combination no.8 $DL + 0.75 \times LL + 0.525 \times E_h + 0.525 \times E_v + 0.75 \times SL$ (0.196)

Load combination no.9 $0.6 \times DL + 0.6 \times W$ (0.365)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b =$ **52.7** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) =$ **358.4** in⁴/ft

Section modulus $S = I / c =$ **94** in³/ft

Radius of gyration $r = \sqrt{[I / A]} =$ **2.608** in

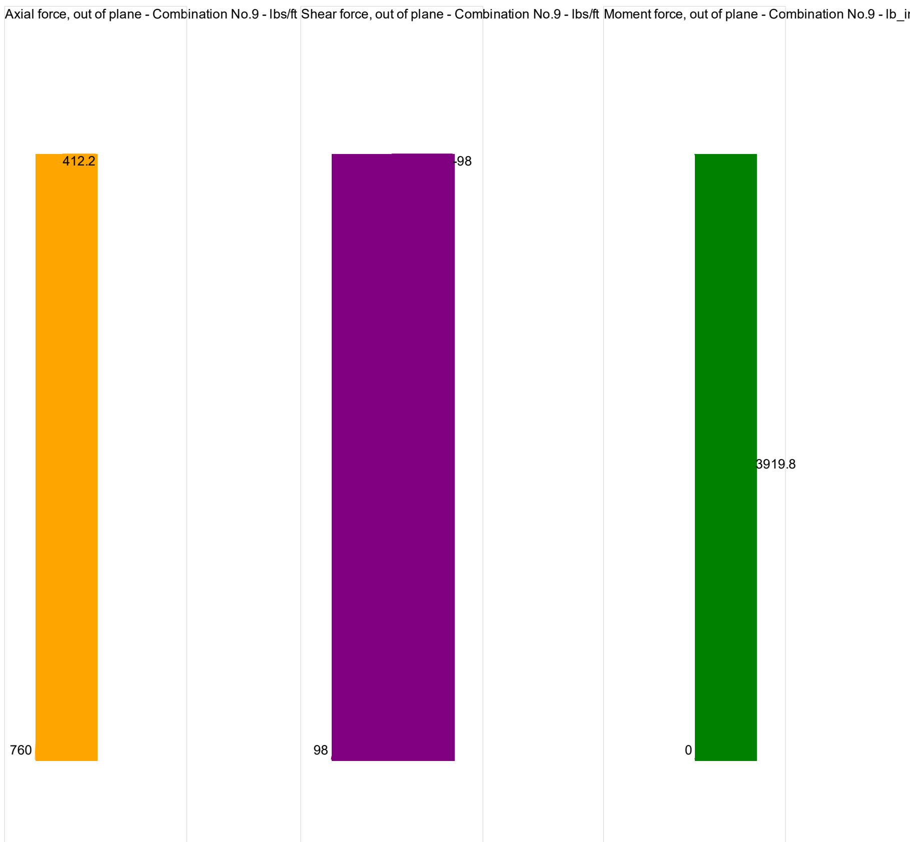
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell9}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell15}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell21}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell22}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell23}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell25}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell26}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell27}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell28}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell29}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell31}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell32}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell33}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell34}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell35}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell37}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell38}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell39}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell40}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell41}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell43}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell44}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell45}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell46}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell47}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell49}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell50}^2) =$ **222107378** in⁴

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{536491}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 6.67 ft

Axial load at max moment loc. of panel $P = \mathbf{586}$ lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{11.1}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{61.355} < 99$

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] = \mathbf{454.5}$ psi

$f_a / F_a = \mathbf{0.024}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] = \mathbf{23918}$ lb/ft

$P / P_a = \mathbf{0.025}$

Bending moment at max. moment loc. of panel $M = \mathbf{3920}$ lb_in/ft

Depth of reinforcement $d = \mathbf{3.813}$ in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{48}$ in

Modular ratio $n = E_s / E_m = \mathbf{14.321}$

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m = \mathbf{1013}$ psi

Balance point $k_{bal} = n / (F_s / F_b + n) = \mathbf{0.312}$

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s = \mathbf{0.001103}$

Project	Job Ref.
I-10 Rest Area	B190988.00

Section	Sheet no./rev.
Bathroom Wall 2	5

Calc. by	Date	Chk'd by	Date	App'd by	Date
ALM	9/3/2020	NJA	9/3/2020		

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal})$ = **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m$ = **1012.5** psi

Tension at balance point $T_{bal} = A_s \times F_s$ = **2454** lb/ft

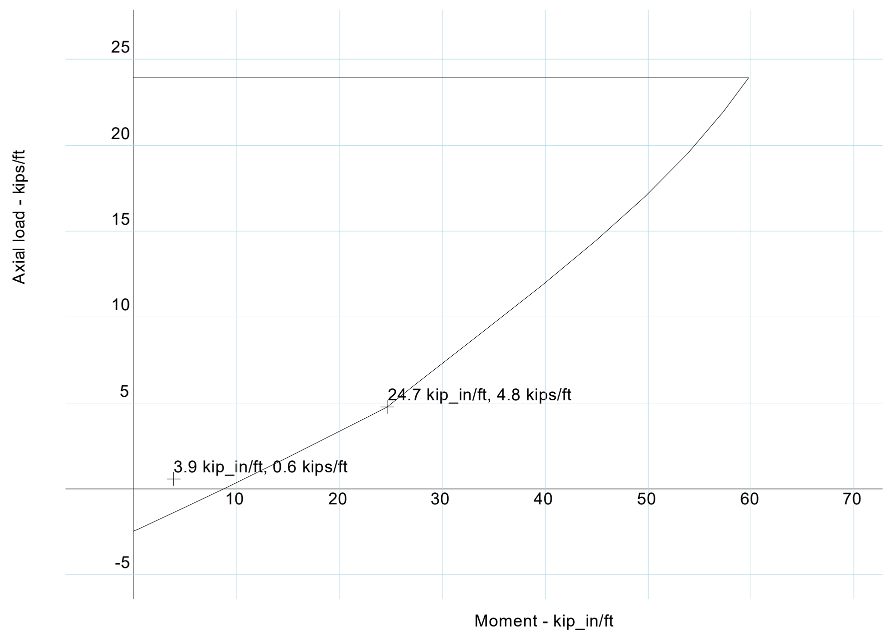
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2$ = **7222** lb/ft

Axial load at balance point $P_{bal} = C_{bal} - T_{bal}$ = **4768** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3)$ = **24673** lb_in/ft

Maximum moment from interaction diagram M_c = **10742** lb_in/ft

M / M_c = **0.365**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force V = **98** lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **7.4** in^2/ft

Shear stress $f_v = V / A_{nv}$ = **13.2** psi

Compressive force $N_v = 0.60 \times P_{DL,t,out} / 1\text{ ft}$ = **412.2** lb/ft

Moment M = **0** lb_in/ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_v / A$ =**96.8** psi

Allowable shear stress $F_v = \min(F_{vm}, 3 \times \sqrt{(f'_m \times 1\text{ psi})})$ = **96.8** psi

Shear utilization f_v / F_v = **0.136**

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.2.07

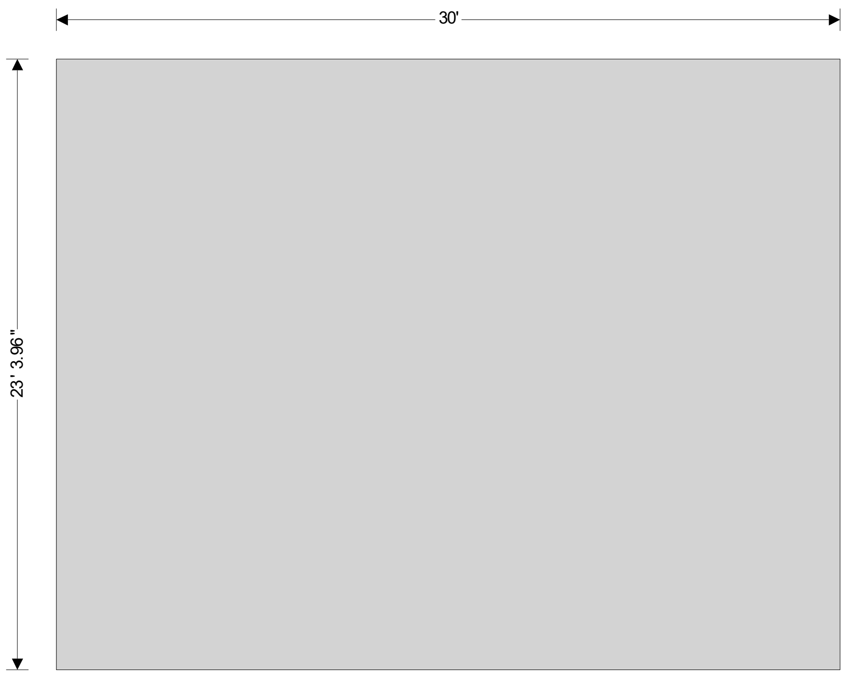
Masonry wall panel details

Bathroom Masonry Walls mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **30** ft

Panel height h = **23.33** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

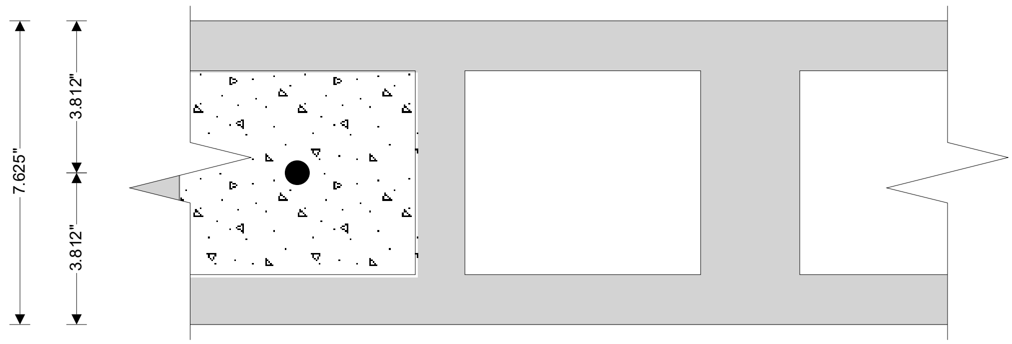
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

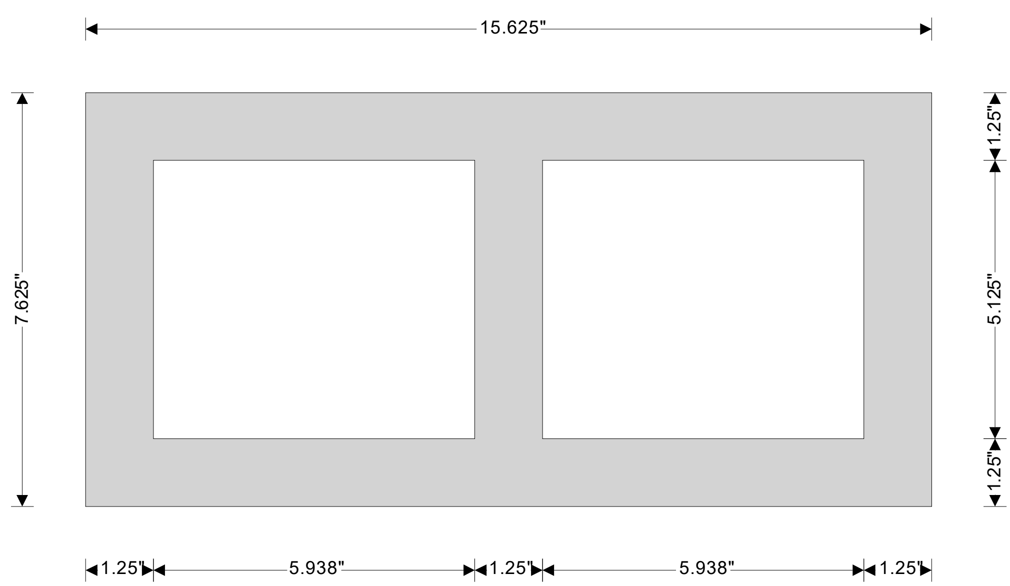
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **1900** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry — f'_m = **1500** psi

Modulus of elasticity for masonry — $E_m = 900 \times f'_m$ = **1350000** psi

Shear modulus of masonry — $G_v = 0.4 \times E_m$ = **540000** psi

From TMS 402 -11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed — F_{t_norm} = **51** psi

Allowable flexural tensile stress parallel to bed — F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement — f_y = **60000** psi

Allowable tensile stress in reinforcement — F_s = **32000** psi

Modulus of elasticity for reinforcement — E_s = **29000000** psi

Vertical reinforcement provided — No.5 bars at 48 in centers

Area of vertical reinforcement — $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in^2/ft

Lateral out-of-plane loads

Wind load on panel — W = **16.3** psf

Wind load on parapet — W_p = **16.3** psf

Seismic load factor (ASCE7 12.11.1) — $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall — $E_{wall} = max(F_p,0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load — E_{add} = **0** psf

Seismic lateral load on panel — $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall — V_W = **23200** lbs

Vertical loading details

Dead load at supported level — DL = **90** lb/ft

Live roof load at supported level — LL_r = **60** lb/ft

Vertical seismic load factor applied to dead load — $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 — DL (0.145)

Load combination no.2 — DL + LL (0.145)

Load combination no.5 — $DL + 0.6 \times W$ (0.732)

Load combination no.7 — $DL + 0.75 \times LL + 0.45 \times W + 0.75 \times (LL_r$ or SL or RL) (0.542)

Load combination no.8 — $DL + 0.75 \times LL + 0.525 \times E_h + 0.525 \times E_v + 0.75 \times SL$ (0.781)

Load combination no.9 — $0.6 \times DL + 0.6 \times W$ (0.798)

Properties of masonry section

Cross-sectional area — $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia — $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in^4/ft

Section modulus — $S = I / c$ = **98.3** in^3/ft

Radius of gyration — $r = \sqrt{[I / A]}$ = **2.495** in

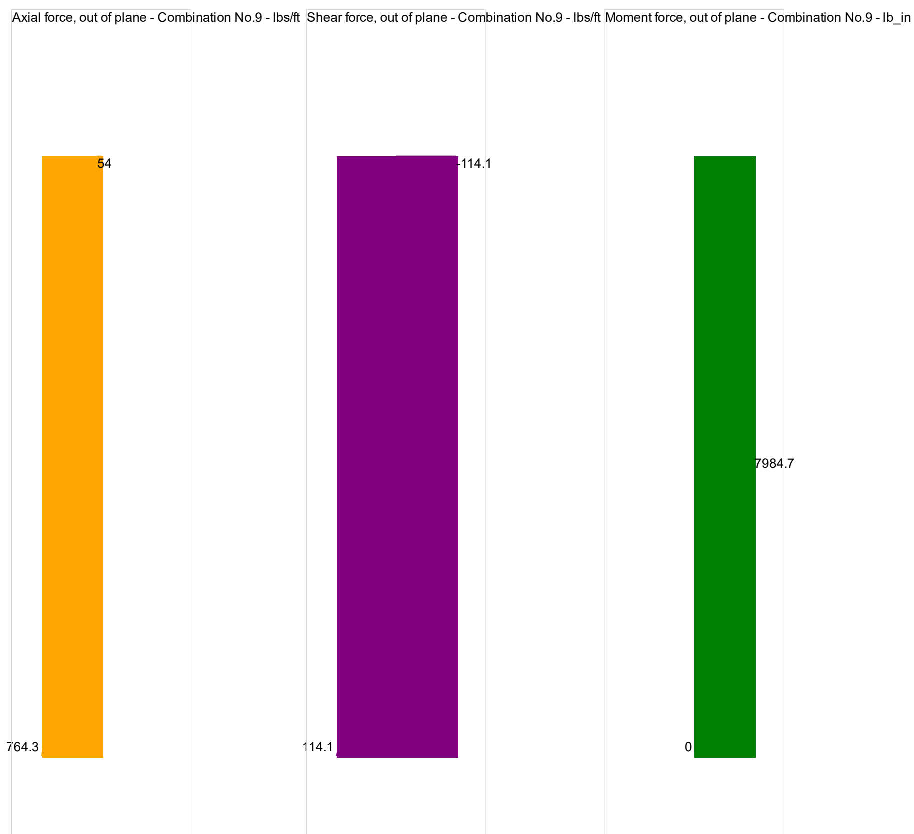
Effective height factor — K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) = \mathbf{21654852}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{120305}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 11.67 ft

Axial load at max moment loc. of panel $P = \mathbf{409}$ lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{6.8}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{112.189}$ > 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{146}$ psi

$f_a / F_a = \mathbf{0.047}$

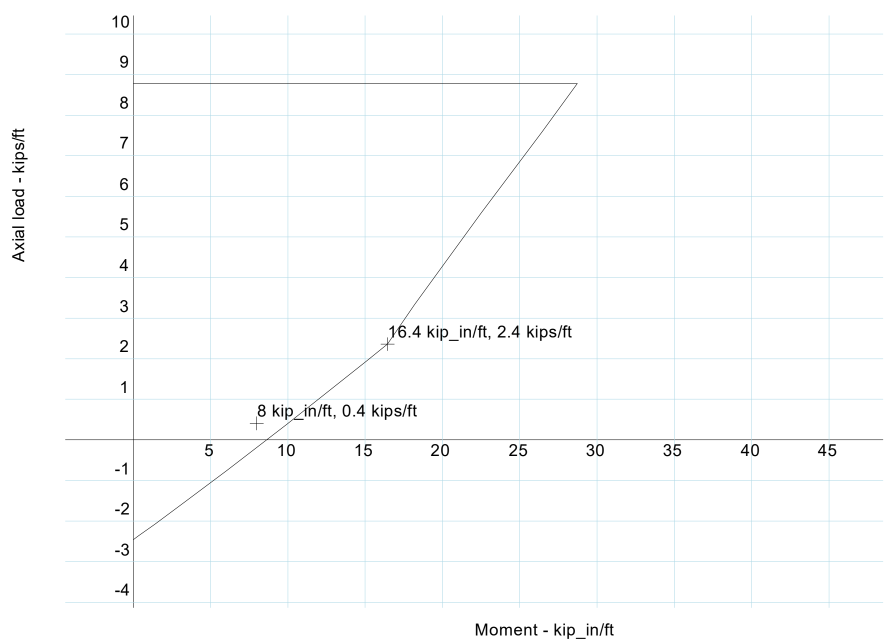
Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{8775}$ lb/ft

$P / P_a = \mathbf{0.047}$

Bending moment at max. moment loc. of panel $M = \mathbf{7985}$ lb_in/ft

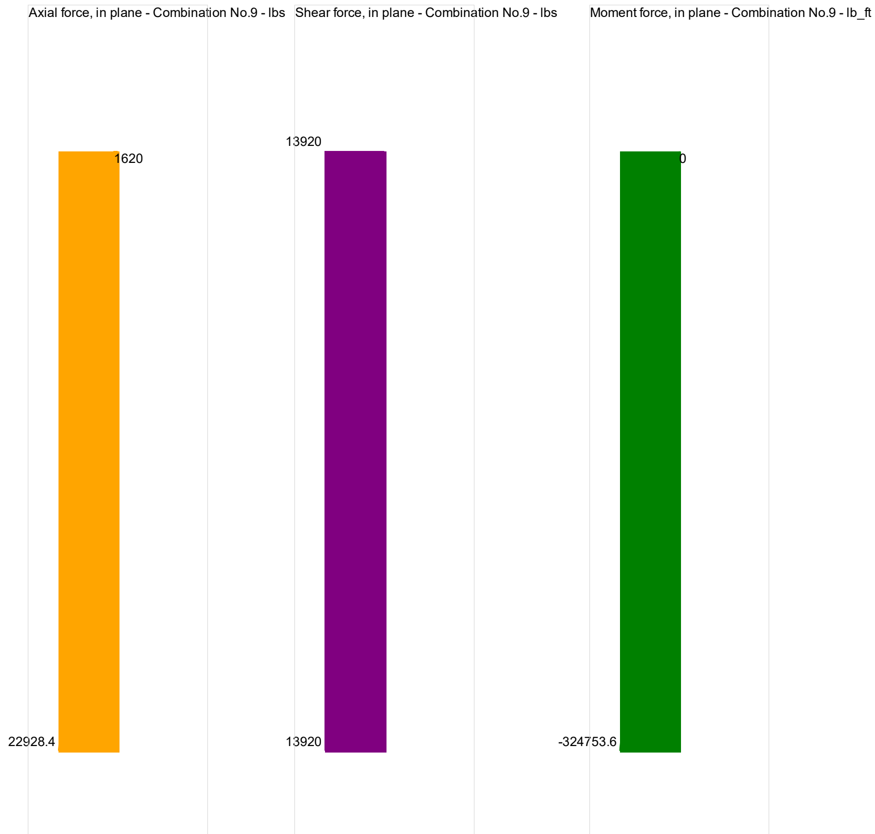
Depth of reinforcement $d = \mathbf{3.813}$ in

Effective width per bar	b_{eff} = min(s, 6 × t_{nom}, 72 in) = **48** in
Modular ratio	n = E_s / E_m = **21.481**
Allowable compressive stress due to flexural load	F_b = (0.45) × f'_m = **675** psi
Balance point	k_{bal} = n / (F_s / F_b + n) = **0.312**
Tensile strain in reinforcement	ε_s = F_s / E_s = **0.001103**
Compressive strain in masonry	ε_m = ε_s × k_{bal} / (1 - k_{bal}) = **0.000500**
Compressive stress at balance point	f_{bal} = ε_m × E_m = **675** psi
Tension at balance point	T_{bal} = A_s × F_s = **2454** lb/ft
Compression at balance point	C_{bal} = k_{bal} × d × f_{bal} / 2 = **4815** lb/ft
Axial load at balance point	P_{bal} = C_{bal} – T_{bal} = **2360** lb/ft
Moment at balance point	M_{bal} = T_{bal} × (d - t / 2) + C_{bal} × (t / 2 – k_{bal} × d / 3) = **16448** lb_in/ft
Maximum moment from interaction diagram	M_c = **10003** lb_in/ft
	M / M_c = **0.798**



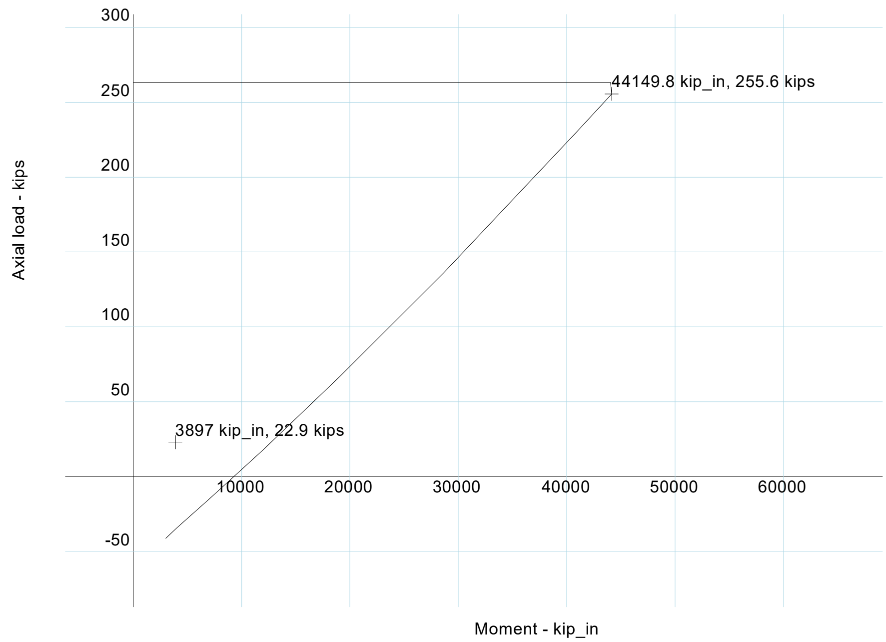
Allowable stress interaction diagram

Consider wall at bottom under load combination no.9



Shear force	V = **13920** lbs
Depth to reinforcement	$d_{in} = L - l_b / 4$ = **29.7** ft
Net shear area	$A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf})$ = **1484.3** in^2
Shear stress	$f_v = V / A_{nv}$ = **9.4** psi
Compressive force	$N_v = 0.6 \times P_{DL,b,in}$ = **22928.4** lbs
Moment	M_v = **324753.6** lb_ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L)$ = **54** psi
Allowable shear stress	$F_v = \min(F_{vm},2.29 \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **54** psi
Shear utilization	f_v / F_v = **0.174**
Axial load	P = **22928** lb
By iteration, tension at balance point	T_{bal} = **30052** lb
Compression at balance point	C_{bal} = **285679** lb
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal}$ = **255627** lb
By iteration, moment at balance point	M_{bal} = **44149779** lb_in
Maximum moment from interaction diagram	M_c = **12712561** lb_in
	M_v / M_c = **0.307**

Project				Job Ref.	
I-10 Rest Area				B190988.00	
Section				Sheet no./rev.	
Bathroom Wall 3, mid/high				7	
Calc. by	Date	Chk'd by	Date	App'd by	Date
ALM	9/3/2020	NJA	9/3/2020		



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.2.07

Masonry wall panel details

Bathroom Masonry Wall 3 end - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length $L = \mathbf{30}$ ft

Panel height $h = \mathbf{13.33}$ ft

Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) $I_e = \mathbf{1}$

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

$S_{DS} = \mathbf{1}$

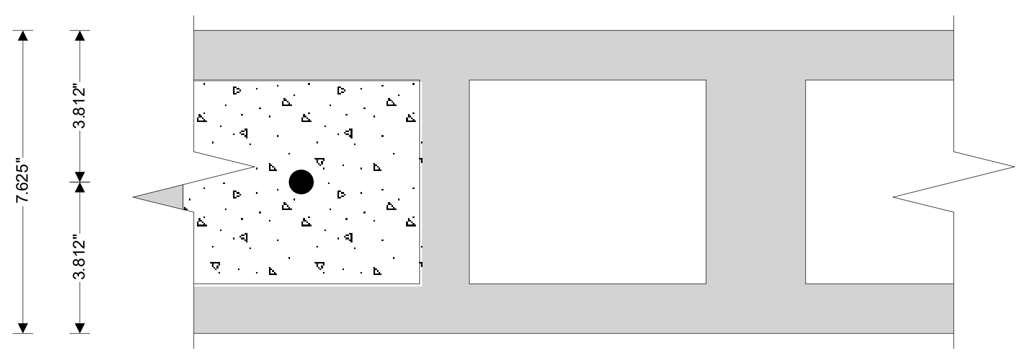
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load $\rho_E = \mathbf{1.0}$

Construction details

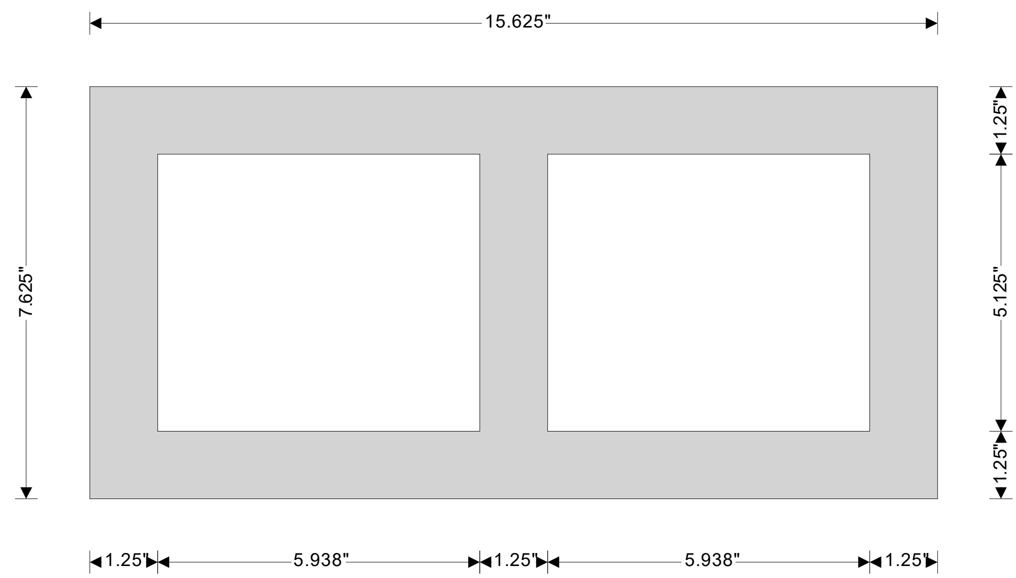
Wall thickness $t = \mathbf{7.625}$ in



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **2800** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry	f'_m = **2000** psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m$ = **1800000** psi
Shear modulus of masonry	$G_v = 0.4 \times E_m$ = **720000** psi

From TMS 402 -11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed	F_{t_norm} = **42** psi
Allowable flexural tensile stress parallel to bed	F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 48 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **22.2** psf

Wind load on parapet W_p = **22.2** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p,0.1) \times w_{SW}$ = **17.4** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **17.4** psf

Lateral in-plane loads

Wind shear load on wall V_W = **23200** lbs

Vertical loading details

Dead load at supported level DL = **90** lb/ft

Live roof load at supported level LL_r = **60** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.031)

Load combination no.2 DL + LL (0.031)

Load combination no.5 DL + 0.6 × W (0.355)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.262)

Load combination no.8 DL + 0.75 × LL + 0.525 × E_h + 0.525 × E_v + 0.75 × SL (0.240)

Load combination no.9 0.6 × DL + 0.6 × W (0.375)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **52.7** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **358.4** in⁴/ft

Section modulus $S = I / c$ = **94** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.608** in

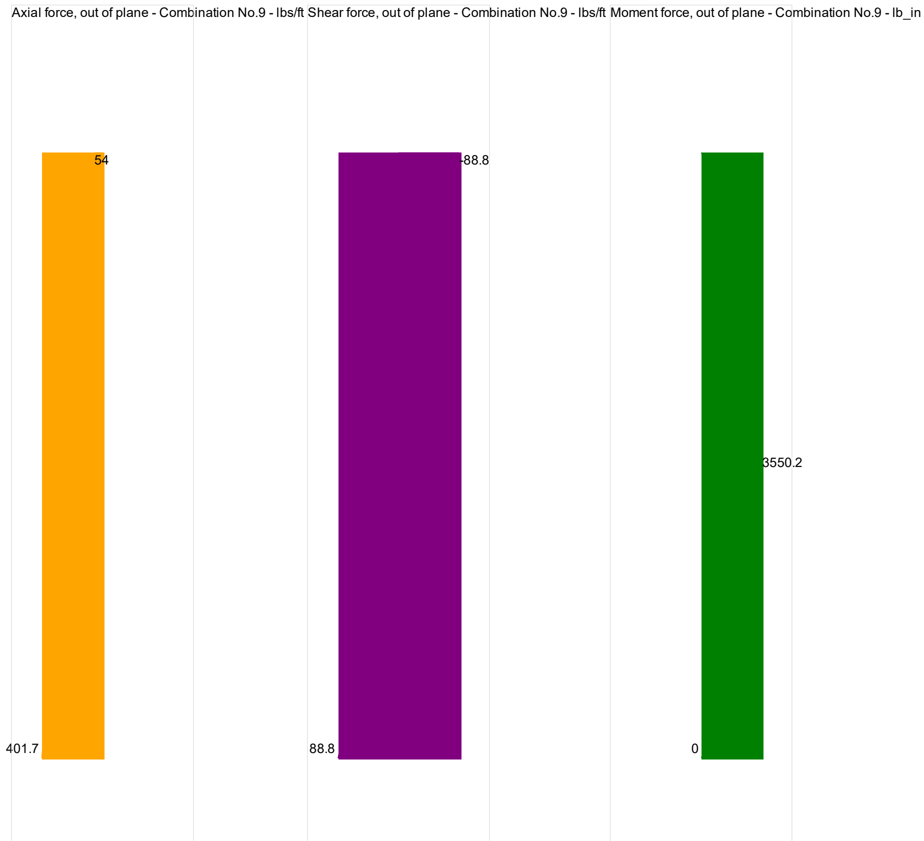
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell9}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell15}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell21}^2)$ = **18709517** in⁴

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2)$ = **103942** in³

Consider wall at maximum moment location under load combination no.9



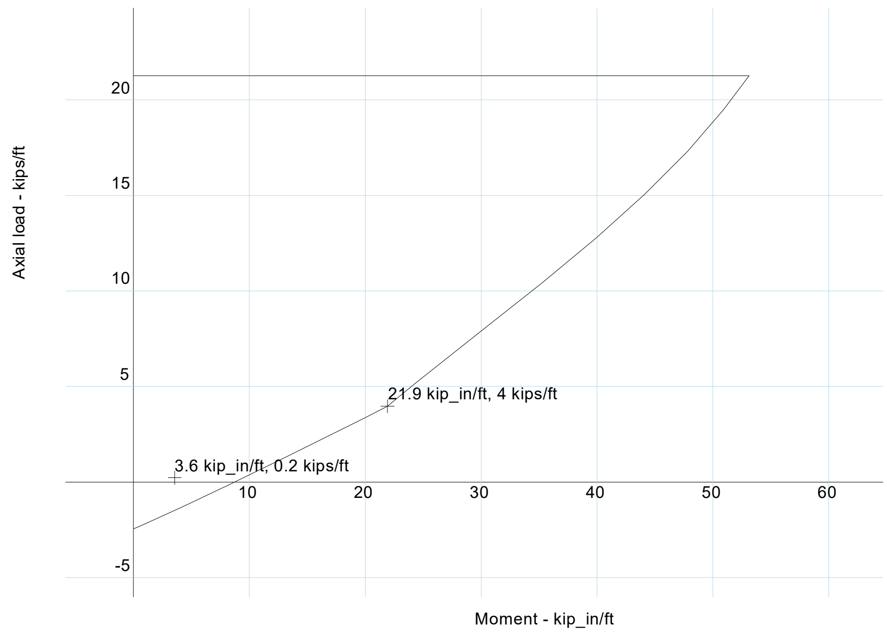
Maximum moment location	6.67 ft
Axial load at max moment loc. of panel	P = **228** lb/ft
Compressive stress due to axial load	$f_a = P / A =$ **4.3** psi
Slenderness ratio	$(K \times h) / r =$ **61.341** < 99
Allowable compressive stress due to axial load	$F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **404** psi
	$f_a / F_a =$ **0.011**
Allowable compressive force	$P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **21263** lb/ft
	$P / P_a =$ **0.011**
Bending moment at max. moment loc. of panel	M = **3550** lb_in/ft
Depth of reinforcement	d = **3.813** in
Effective width per bar	$b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) =$ **48** in
Modular ratio	$n = E_s / E_m =$ **16.111**
Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m =$ **900** psi
Balance point	$k_{bal} = n / (F_s / F_b + n) =$ **0.312**
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s =$ **0.001103**
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m =$ **900** psi
Tension at balance point	$T_{bal} = A_s \times F_s =$ **2454** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6420** lb/ft

Axial load at balance point $P_{bal} = C_{bal} – T_{bal}$ = **3965** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3)$ = **21931** lb_in/ft

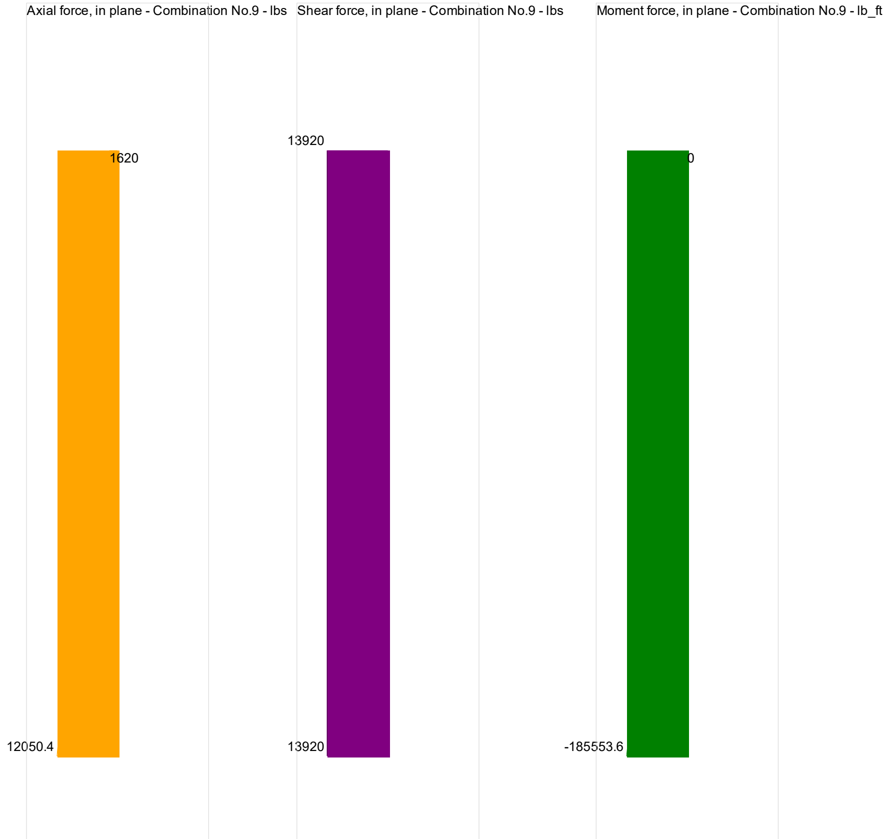
Maximum moment from interaction diagram M_c = **9479** lb_in/ft

M / M_c = **0.375**

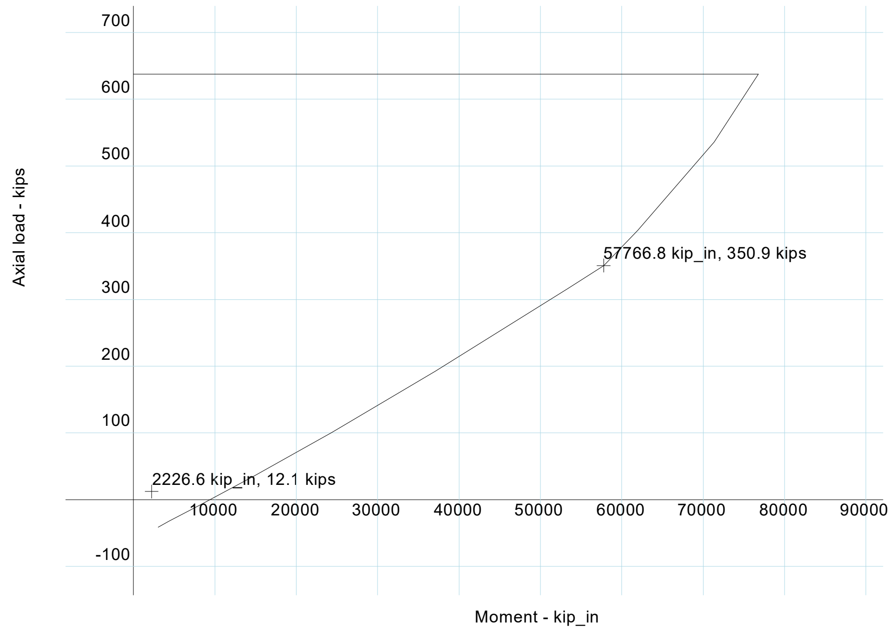


Allowable stress interaction diagram

Consider wall at bottom under load combination no.9



Shear force	V = **13920** lbs
Depth to reinforcement	d_{in} = L - l_b / 4 = **29.7** ft
Net shear area	$A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf})$ = **1187.3** in^2
Shear stress	$f_v = V / A_{nv}$ = **11.7** psi
Compressive force	$N_v = 0.6 \times P_{DL,b,in}$ = **12050.4** lbs
Moment	M_v = **185553.6** lb_ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L)$ = **73.8** psi
Allowable shear stress	$F_v = \min(F_{vm},2.73 \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **73.8** psi
Shear utilization	f_v / F_v = **0.159**
Axial load	P = **12050** lb
By iteration, tension at balance point	T_{bal} = **30052** lb
Compression at balance point	C_{bal} = **380906** lb
Axial load at balance point	$P_{bal} = C_{bal} – T_{bal}$ = **350853** lb
By iteration, moment at balance point	M_{bal} = **57766814** lb_in
Maximum moment from interaction diagram	M_c = **11265710** lb_in
	M_v / M_c = **0.198**



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

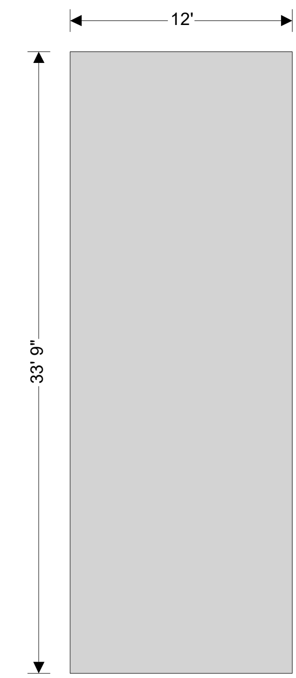
Masonry wall panel details

Shear wall 4 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **12** ft

Panel height h = **33.75** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

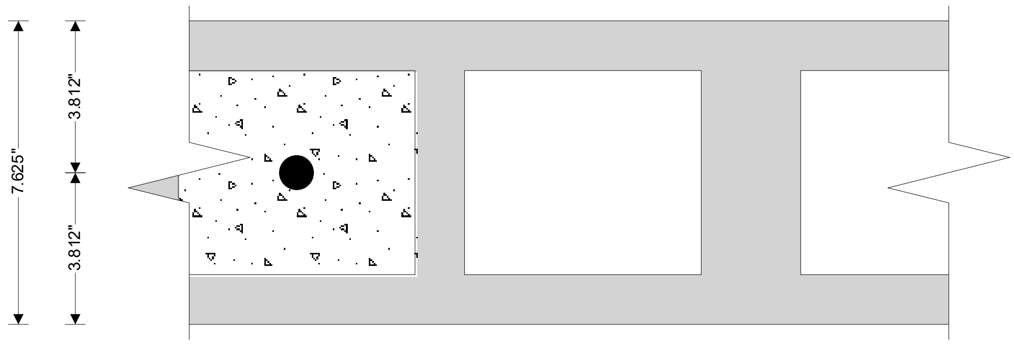
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

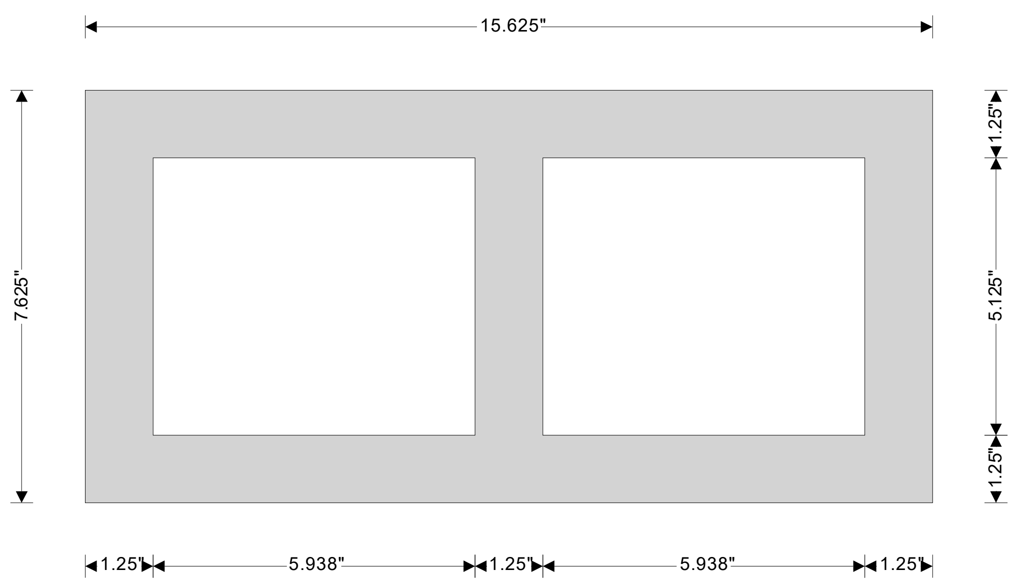
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **1900** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1900** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **44** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.7 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.30** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p,0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall V_W = **14300** lbs

Vertical loading details

Dead load at supported level DL = **1292** lb/ft at an eccentricity of 3.8 in

Live load at supported level LL = **820** lb/ft at an eccentricity of 3.8 in

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.568)

Load combination no.2 DL + LL (0.723)

Load combination no.5 DL + 0.6 × W (0.840)

Load combination no.9 0.6 × DL + 0.6 × W (0.910)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in⁴/ft

Section modulus $S = I / c$ = **98.3** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

Effective height factor K = **1**

Properties for walls loaded in-plane:

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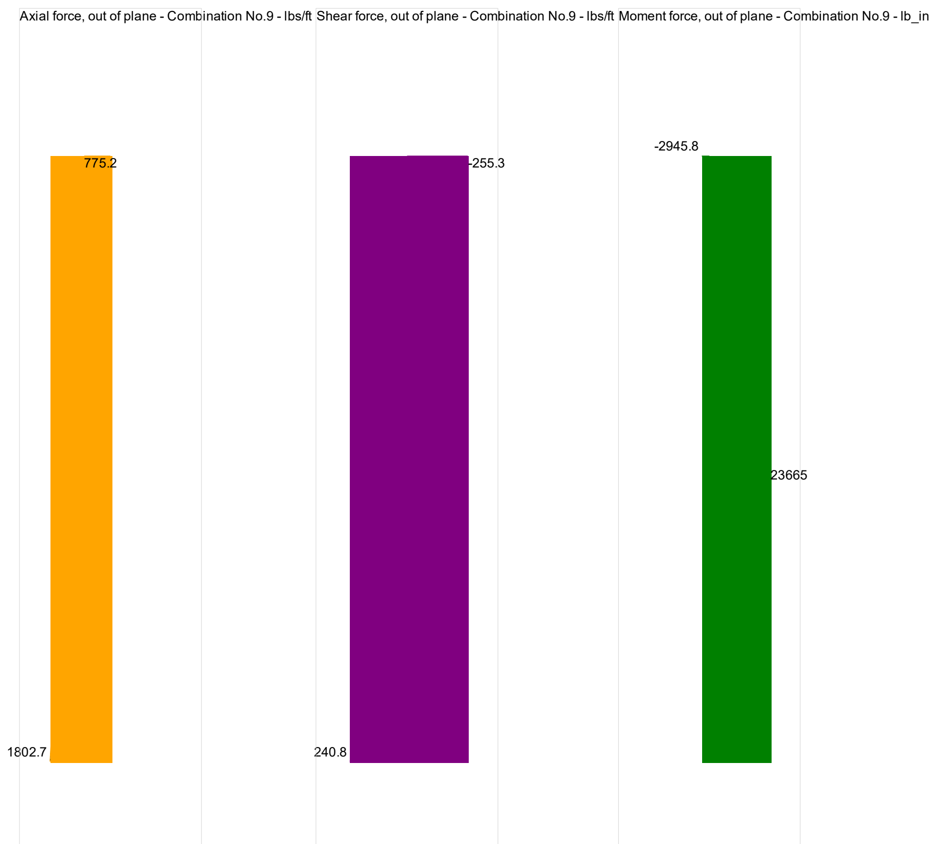
Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) = \mathbf{1276966}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{17736}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 16.38 ft

Axial load at max moment loc. of panel P = **1304** lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{21.7}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{162.296}$ > 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{88.4}$ psi

$f_a / F_a = \mathbf{0.245}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{5292}$ lb/ft

$P / P_a = \mathbf{0.246}$

Bending moment at max. moment loc. of panel M = **23665** lb_in/ft

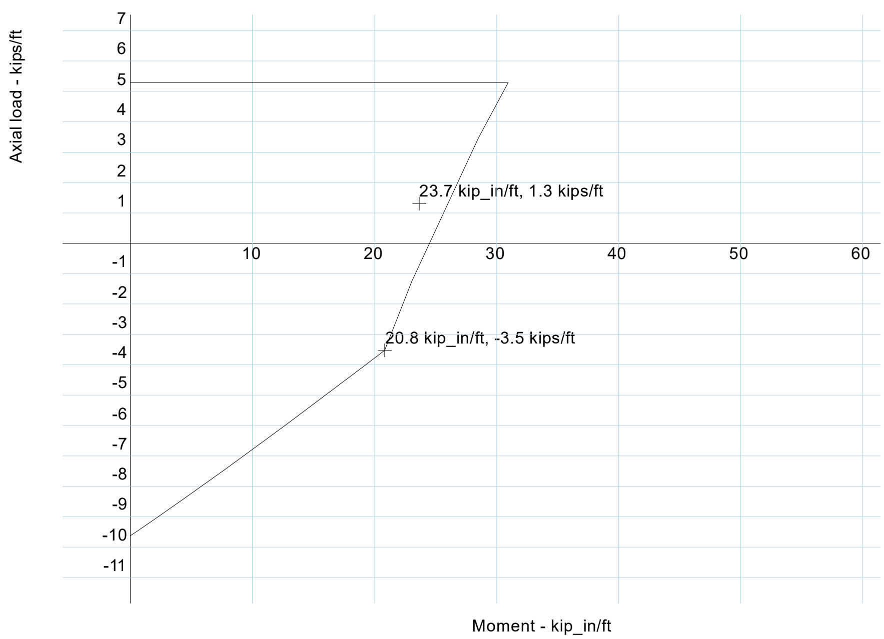
Depth of reinforcement d = **3.813** in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{24}$ in

Modular ratio $n = E_s / E_m = \mathbf{16.959}$

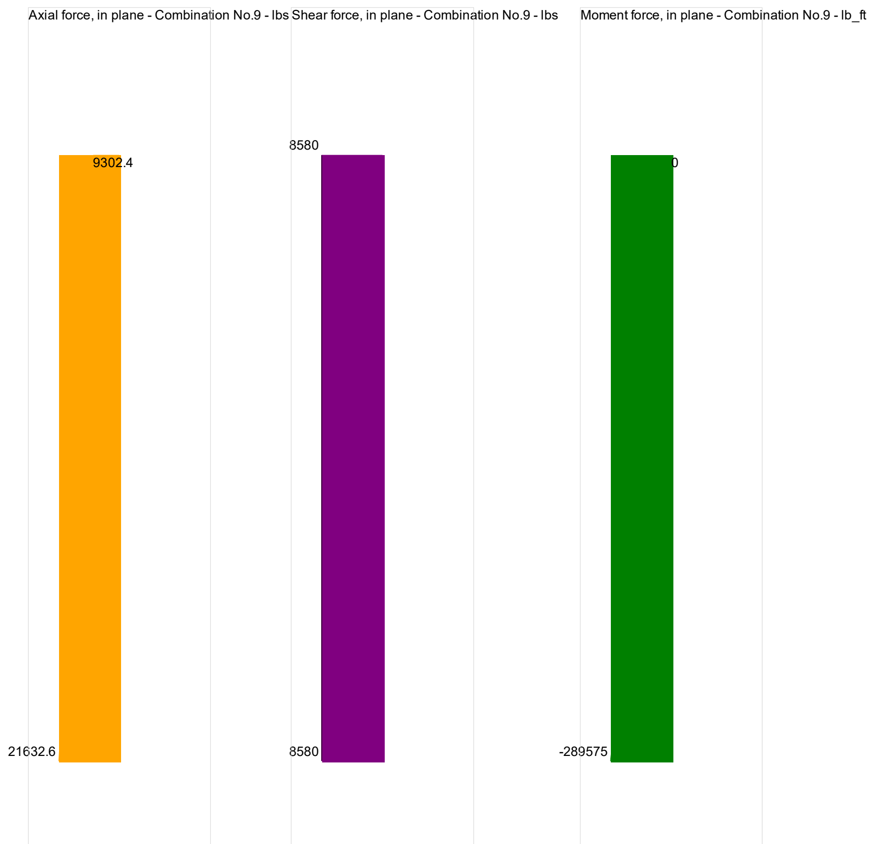
Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Shear wall 4				Sheet no./rev. 5	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date

Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m = \mathbf{855}$ psi
Balance point	$k_{bal} = n / (F_s / F_b + n) = \mathbf{0.312}$
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s = \mathbf{0.001103}$
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) = \mathbf{0.000500}$
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m = \mathbf{855}$ psi
Tension at balance point	$T_{bal} = A_s \times F_s = \mathbf{9621}$ lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 = \mathbf{6099}$ lb/ft
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal} = \mathbf{-3522}$ lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3) = \mathbf{20835}$ lb_in/ft
Maximum moment from interaction diagram	$M_c = \mathbf{26011}$ lb_in/ft
	$M / M_c = \mathbf{0.91}$

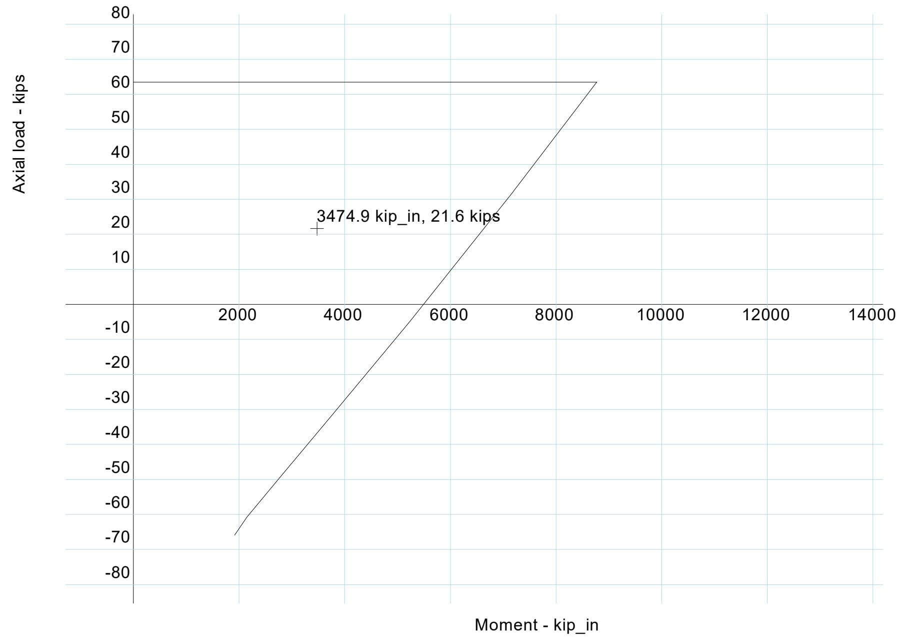


Allowable stress interaction diagram

Consider wall at bottom under load combination no.9



Shear force	V = **8580** lbs
Depth to reinforcement	$d_{in} = L - l_b / 4$ = **11.7** ft
Net shear area	$A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf})$ = **584** in^2
Shear stress	$f_v = V / A_{nv}$ = **14.7** psi
Compressive force	$N_v = 0.6 \times P_{DL,b,in}$ = **21632.6** lbs
Moment	M_v = **289575** lb_ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L)$ = **56.5** psi
Allowable shear stress	$F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times 0.75$ = **42.4** psi
Shear utilization	f_v / F_v = **0.347**
Axial load	P = **21633** lb
By iteration, tension at balance point	T_{bal} = **48277** lb
Compression at balance point	C_{bal} = **142305** lb
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal}$ = **94027** lb
By iteration, moment at balance point	M_{bal} = **10291082** lb_in
Maximum moment from interaction diagram	M_c = **6631953** lb_in
	M_v / M_c = **0.524**



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

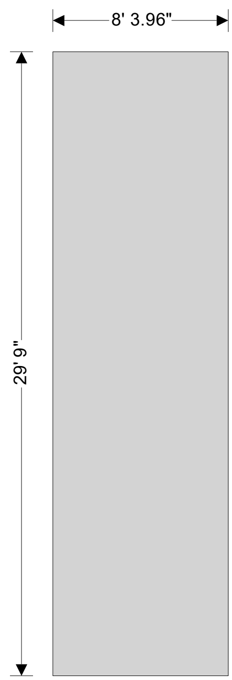
Masonry wall panel details

Shear wall 5 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **8.33** ft

Panel height h = **29.75** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in

Tekla Tedds	Project I-10 Rest Area			Job Ref. B190988.00	
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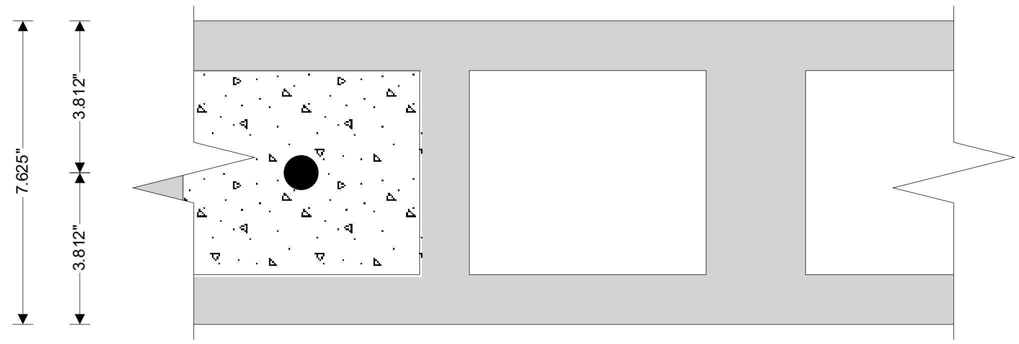
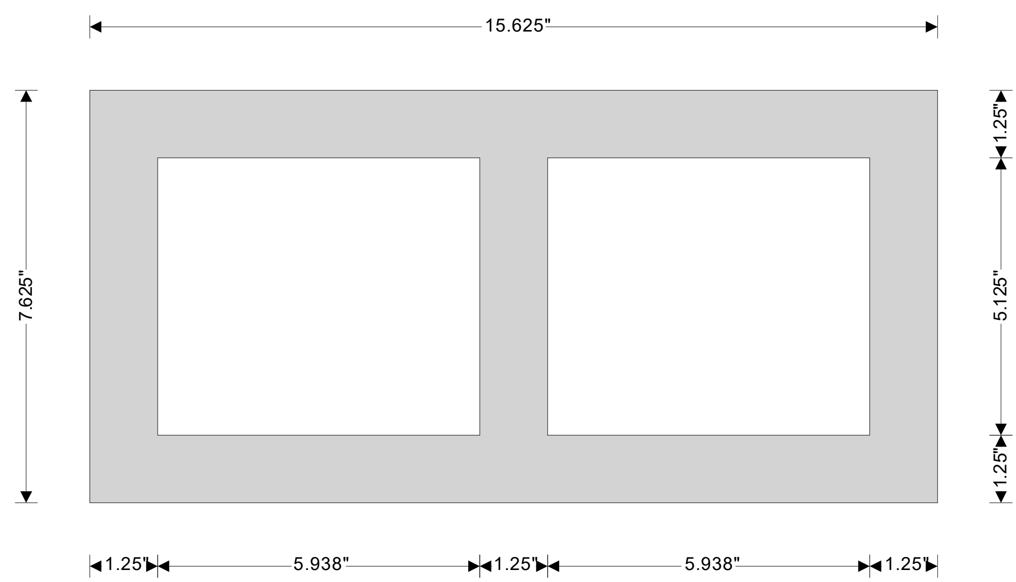


Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **1900** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry — f'_m = **1900** psi

Modulus of elasticity for masonry — $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry — $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed — F_{t_norm} = **44** psi

Allowable flexural tensile stress parallel to bed — F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement — f_y = **60000** psi

Allowable tensile stress in reinforcement — F_s = **32000** psi

Modulus of elasticity for reinforcement — E_s = **29000000** psi

Vertical reinforcement provided — No.7 bars at 24 in centers

Area of vertical reinforcement — $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.30** in^2/ft

Lateral out-of-plane loads

Wind load on panel — W = **24.5** psf

Wind load on parapet — W_p = **24.5** psf

Seismic load factor (ASCE7 12.11.1) — $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall — $E_{wall} = \max(F_p,0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load — E_{add} = **0** psf

Seismic lateral load on panel — $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall — V_W = **14300** lbs

Vertical loading details

Dead load at supported level — DL = **1290** lb/ft at an eccentricity of 3.8 in

Live load at supported level — LL = **820** lb/ft at an eccentricity of 3.8 in

Vertical seismic load factor applied to dead load — $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 — DL (0.411)

Load combination no.2 — DL + LL (0.532)

Load combination no.4 — DL + 0.75 × LL + 0.75 × (LL_r or SL or RL) (0.501)

Load combination no.5 — DL + 0.6 × W (0.823)

Load combination no.7 — DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.589)

Load combination no.9 — 0.6 × DL + 0.6 × W (0.905)

Properties of masonry section

Cross-sectional area — $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia — $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in^4/ft

Section modulus — $S = I / c$ = **98.3** in^3/ft

Radius of gyration — $r = \sqrt{[I / A]}$ = **2.495** in

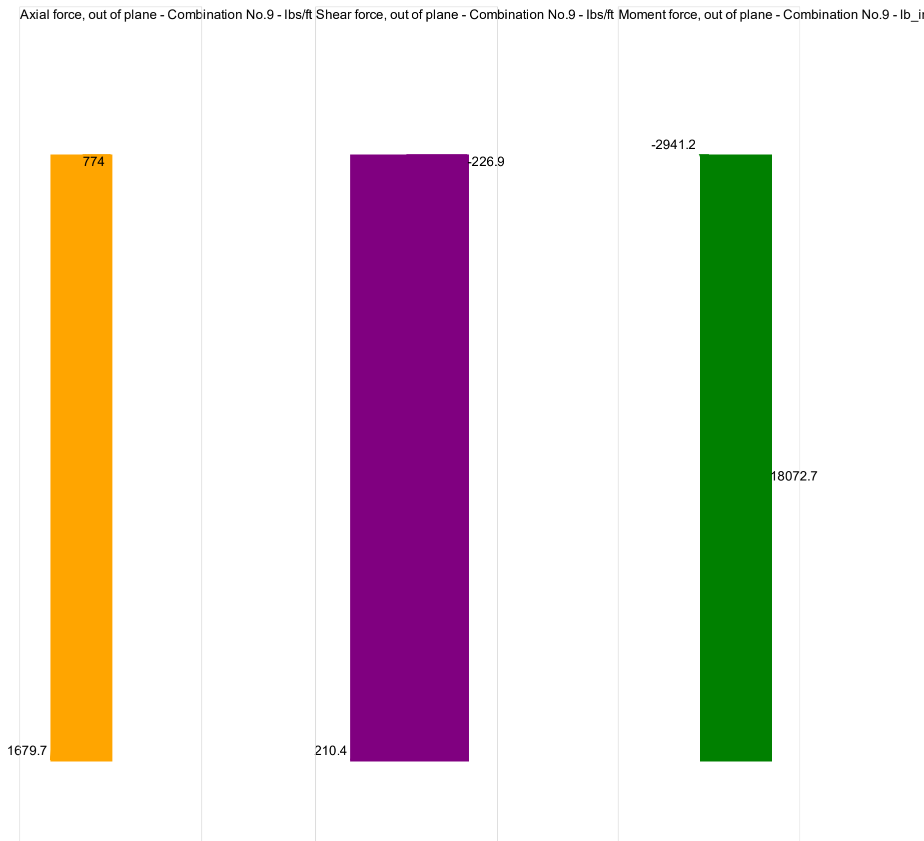
Effective height factor — K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) = \mathbf{454769}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{9099}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 14.31 ft

Axial load at max moment loc. of panel $P = \mathbf{1244}$ lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{20.7}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{143.061}$ > 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{113.7}$ psi

$f_a / F_a = \mathbf{0.182}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{6810}$ lb/ft

$P / P_a = \mathbf{0.183}$

Bending moment at max. moment loc. of panel $M = \mathbf{18073}$ lb_in/ft

Depth of reinforcement $d = \mathbf{3.813}$ in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{24}$ in

Modular ratio $n = E_s / E_m = \mathbf{16.959}$

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m = \mathbf{855}$ psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m =$ **855** psi

Tension at balance point $T_{bal} = A_s \times F_s =$ **9621** lb/ft

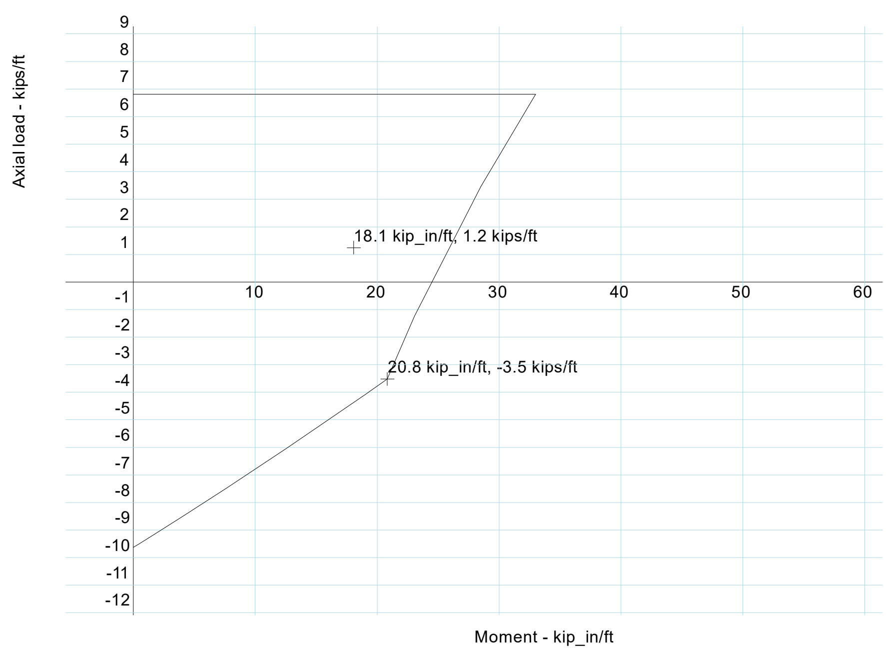
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6099** lb/ft

Axial load at balance point $P_{bal} = C_{bal} – T_{bal} =$ **-3522** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **20835** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **25941** lb_in/ft

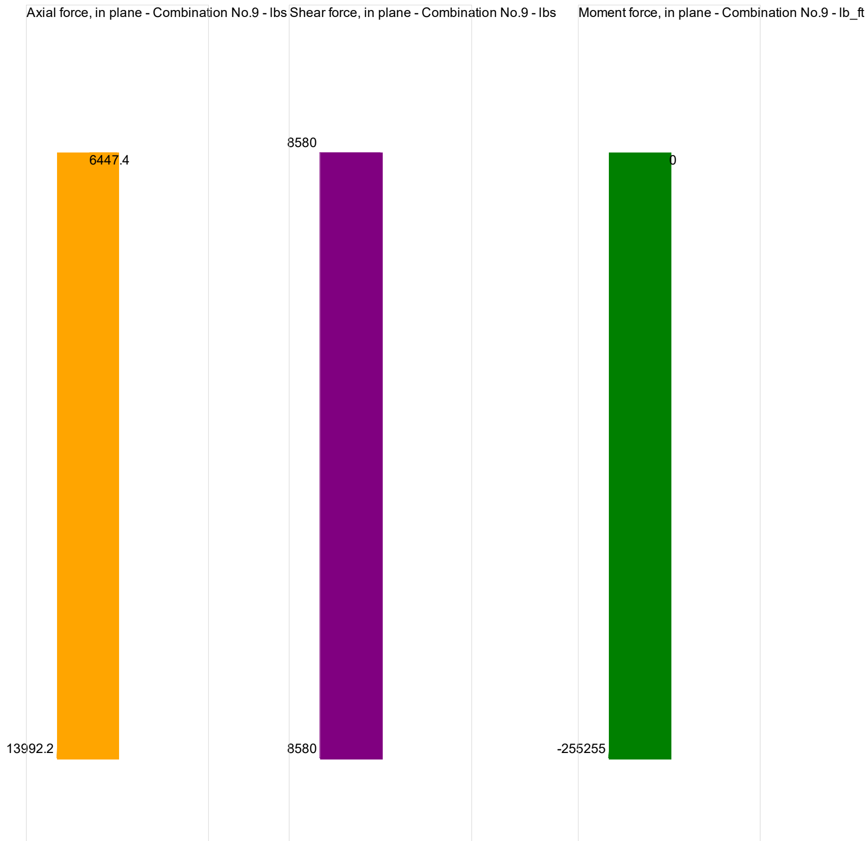
$M / M_c =$ **0.697**



Allowable stress interaction diagram

Consider wall at bottom under load combination no.9

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Shear wall 5				Sheet no./rev. 6	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Shear force $V = \mathbf{8580}$ lbs

Depth to reinforcement $d_{in} = L - l_b / 4 = \mathbf{8}$ ft

Net shear area $A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf}) = \mathbf{400.4}$ in^2

Shear stress $f_v = V / A_{nv} = \mathbf{21.4}$ psi

Compressive force $N_v = 0.6 \times P_{DL,b,in} = \mathbf{13992.2}$ lbs

Moment $M_v = \mathbf{255255}$ lb_ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L) = \mathbf{56}$ psi

Allowable shear stress $F_v = \min(F_{vm},2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times 0.75 = \mathbf{42}$ psi

Shear utilization $f_v / F_v = \mathbf{0.510}$

Axial load $P = \mathbf{13992}$ lb

By iteration, tension at balance point $T_{bal} = \mathbf{36747}$ lb

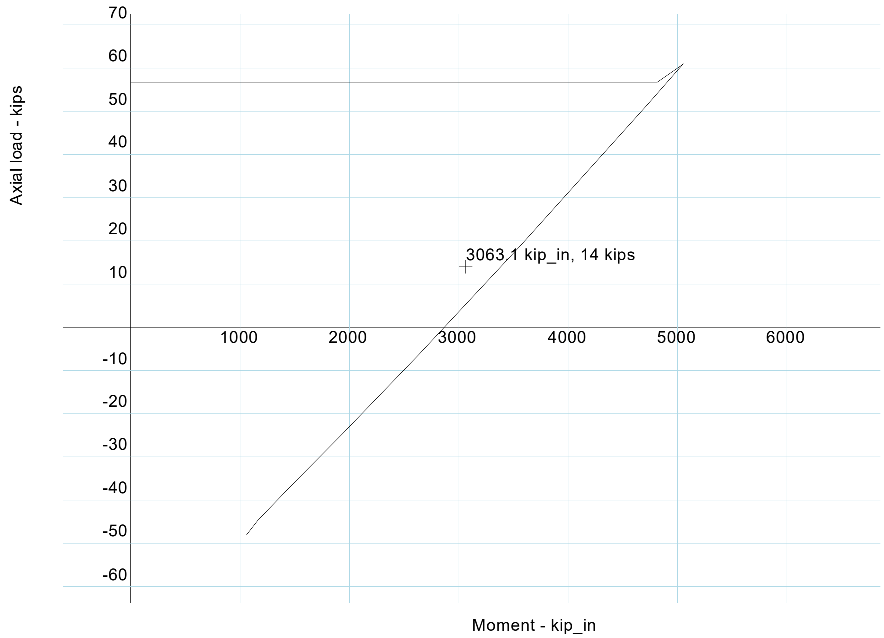
Compression at balance point $C_{bal} = \mathbf{97540}$ lb

Axial load at balance point $P_{bal} = C_{bal} - T_{bal} = \mathbf{60793}$ lb

By iteration, moment at balance point $M_{bal} = \mathbf{5045514}$ lb_in

Maximum moment from interaction diagram $M_c = \mathbf{3382801}$ lb_in

$M_v / M_c = \mathbf{0.905}$



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

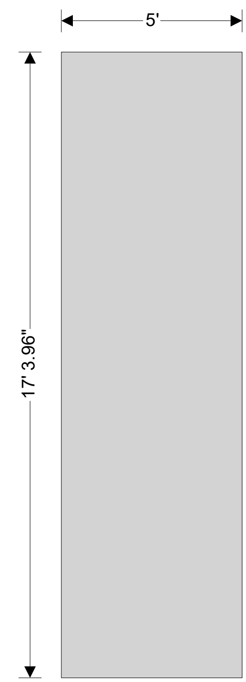
Masonry wall panel details

Shear wall 6 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **5** ft

Panel height h = **17.33** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

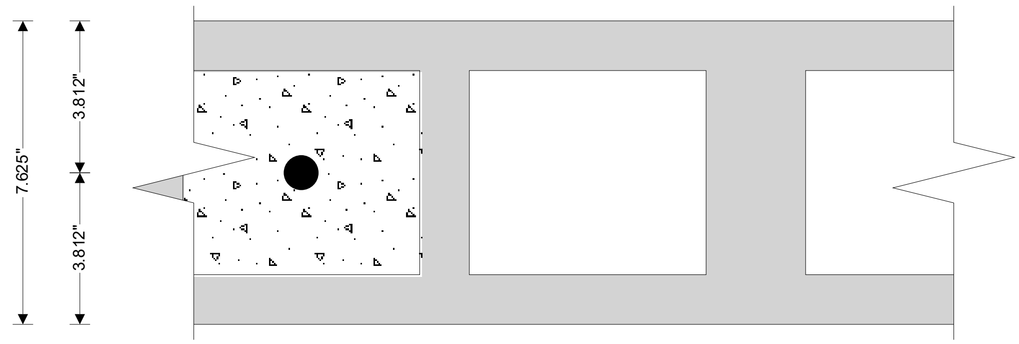
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

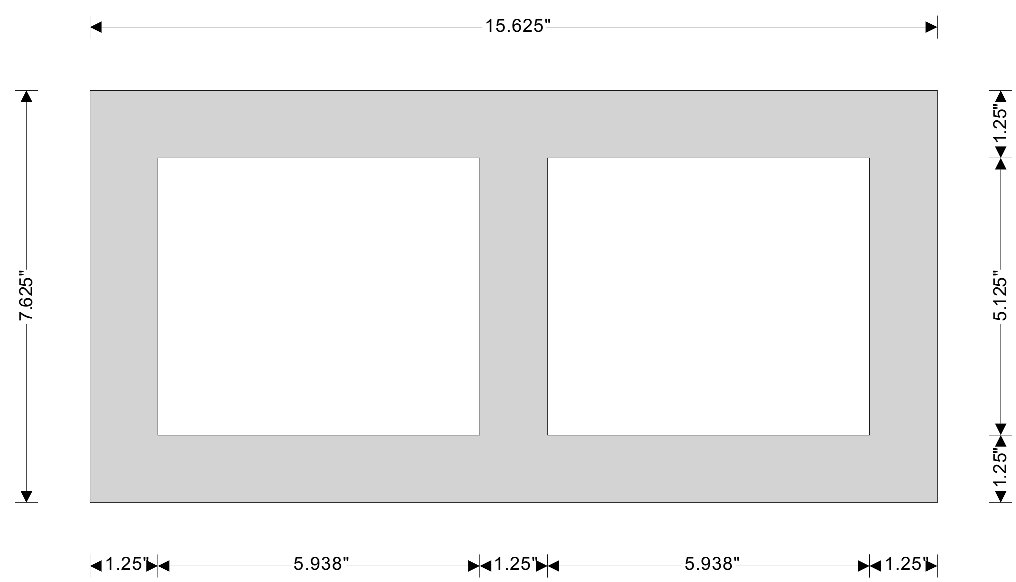
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1900** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **44** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.7 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.30** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall V_W = **5500** lbs

Vertical loading details

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.048)

Load combination no.2 DL + LL (0.048)

Load combination no.3 DL + (LL_r or SL or RL) (0.048)

Load combination no.4 DL + 0.75 × LL + 0.75 × (LL_r or SL or RL) (0.048)

Load combination no.5 DL + 0.6 × W (0.516)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.387)

Load combination no.9 0.6 × DL + 0.6 × W (0.532)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in⁴/ft

Section modulus $S = I / c$ = **98.3** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

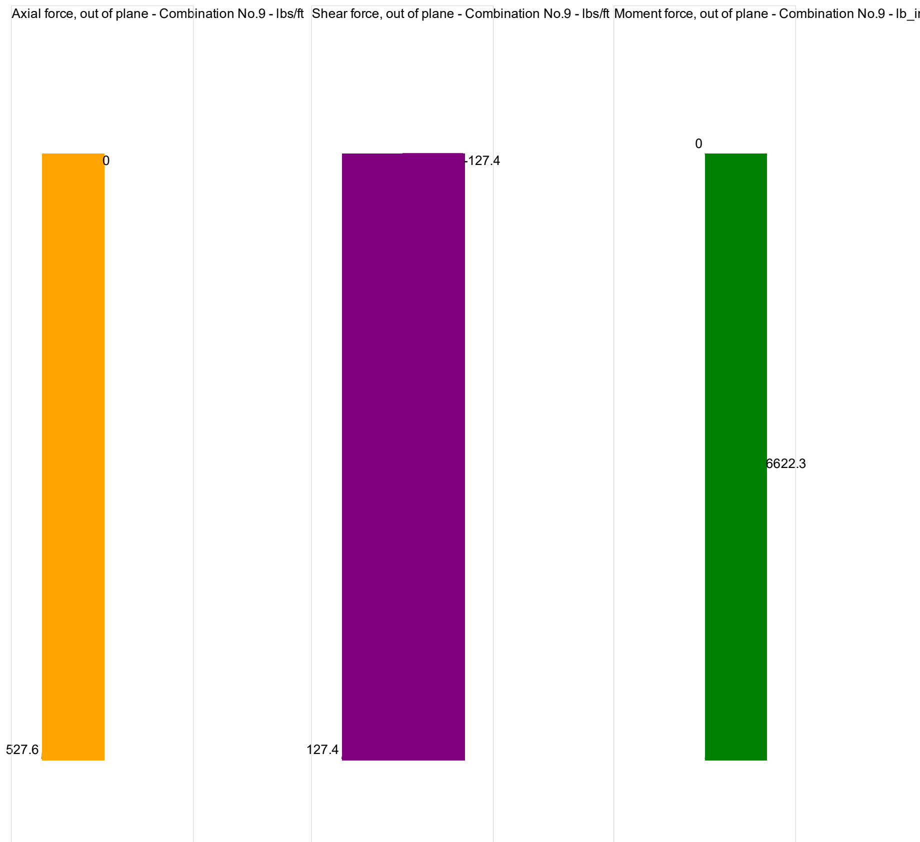
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) =$ **117417** in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) =$ **3914** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 8.66 ft

Axial load at max moment loc. of panel P = **264** lb/ft

Compressive stress due to axial load $f_a = P / A =$ **4.4** psi

Slenderness ratio $(K \times h) / r =$ **83.336** < 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **306.7** psi

$f_a / F_a =$ **0.014**

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **18366** lb/ft

$P / P_a =$ **0.014**

Bending moment at max. moment loc. of panel M = **6622** lb_in/ft

Depth of reinforcement d = **3.813** in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72\ in) =$ **24** in

Modular ratio $n = E_s / E_m =$ **16.959**

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m =$ **855** psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m =$ **855** psi

Tension at balance point $T_{bal} = A_s \times F_s =$ **9621** lb/ft

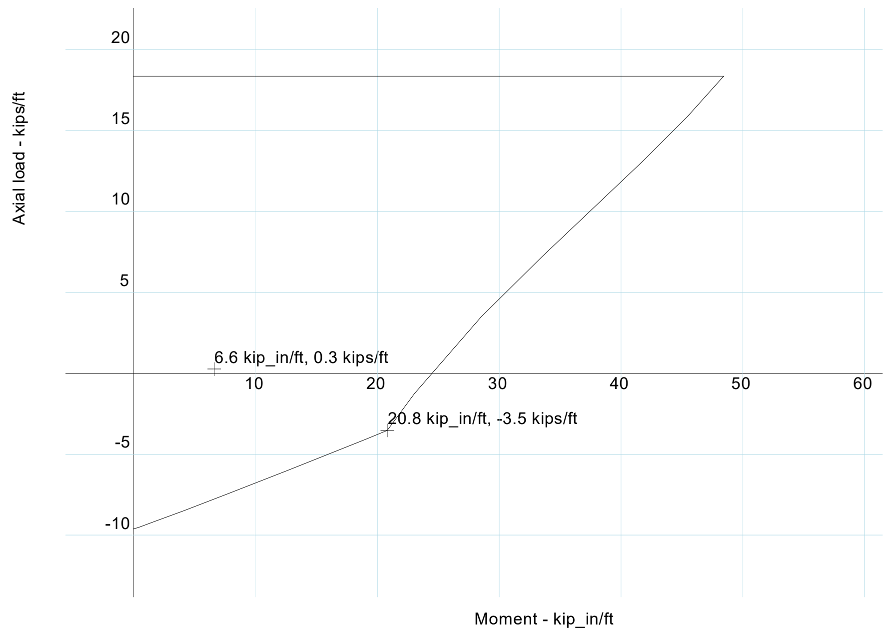
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6099** lb/ft

Axial load at balance point $P_{bal} = C_{bal} - T_{bal} =$ **-3522** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3) =$ **20835** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **24807** lb_in/ft

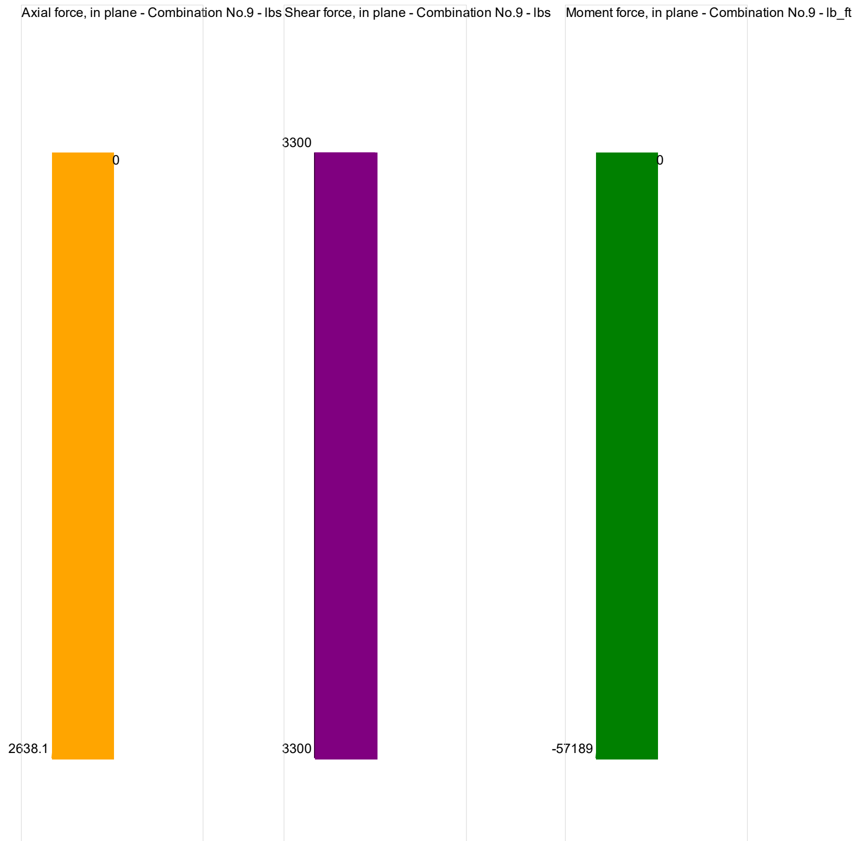
$M / M_c =$ **0.267**



Allowable stress interaction diagram

Consider wall at bottom under load combination no.9

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Shear wall 6				Sheet no./rev. 6	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Shear force V = **3300** lbs

Depth to reinforcement $d_{in} = L - l_b / 4 =$ **4.7** ft

Net shear area $A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf}) =$ **233.8** in^2

Shear stress $f_v = V / A_{nv} =$ **14.1** psi

Compressive force $N_v = 0.6 \times P_{DL,b,in} =$ **2638.1** lbs

Moment $M_v =$ **57189** lb_ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L) =$ **51.2** psi

Allowable shear stress $F_v = \min(F_{vm}, 2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times 0.75 =$ **38.4** psi

Shear utilization $f_v / F_v =$ **0.367**

Axial load P = **2638** lb

By iteration, tension at balance point T_{bal} = **26501** lb

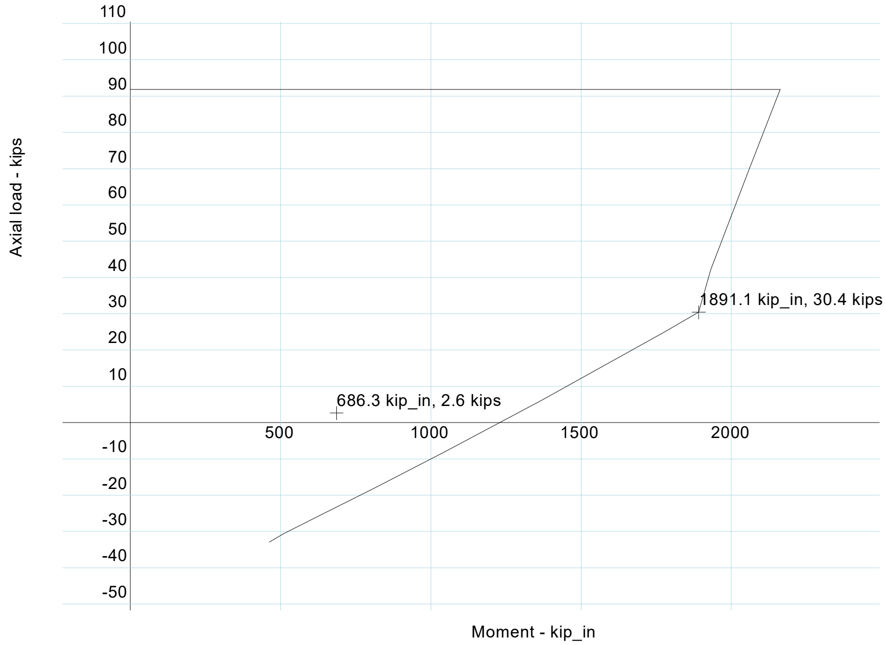
Compression at balance point C_{bal} = **56922** lb

Axial load at balance point $P_{bal} = C_{bal} - T_{bal} =$ **30421** lb

By iteration, moment at balance point M_{bal} = **1891142** lb_in

Maximum moment from interaction diagram M_c = **1289179** lb_in

$M_v / M_c =$ **0.532**



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

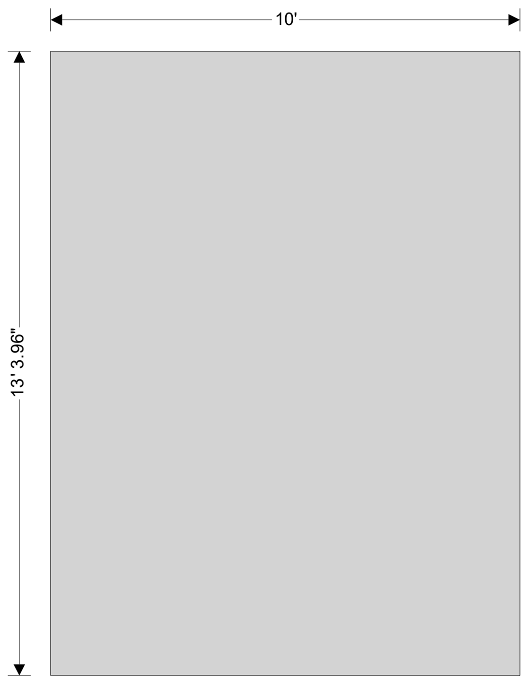
Masonry wall panel details

I-10 Shear wall 7 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **10** ft

Panel height h = **13.33** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

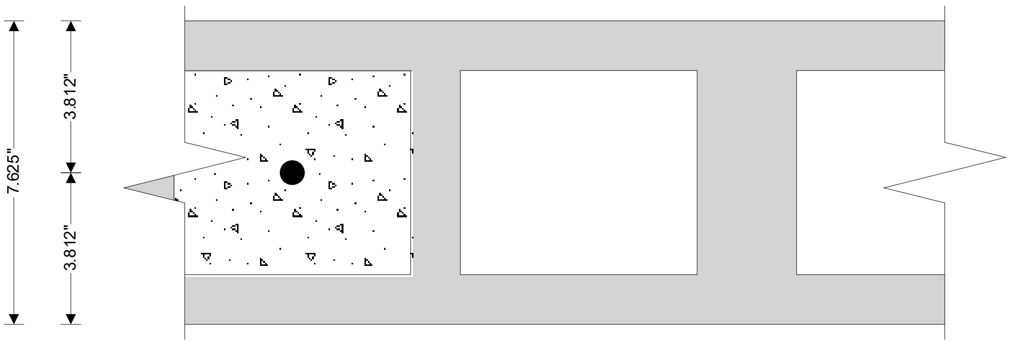
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 40 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

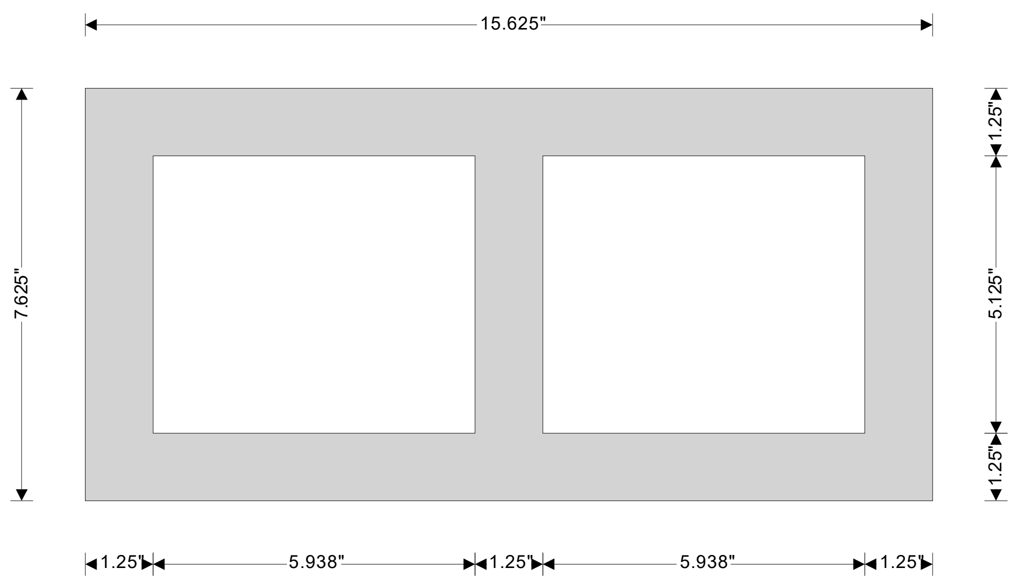
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.2 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **9.35** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **44.83** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1900** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **39** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 40 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.09** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **24.5** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **17.9** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **17.9** psf

Lateral in-plane loads

Wind shear load on wall V_W = **8** lbs

Vertical loading details

Dead load from above DL_{above} = **510** lb/ft

Live roof load at supported level LL_r = **340** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.054)

Load combination no.2 DL + LL (0.054)

Load combination no.5 DL + 0.6 × W (0.299)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.211)

Load combination no.9 0.6 × DL + 0.6 × W (0.326)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.8 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **54.1** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.8 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **361.5** in⁴/ft

Section modulus $S = I / c$ = **94.8** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.585** in

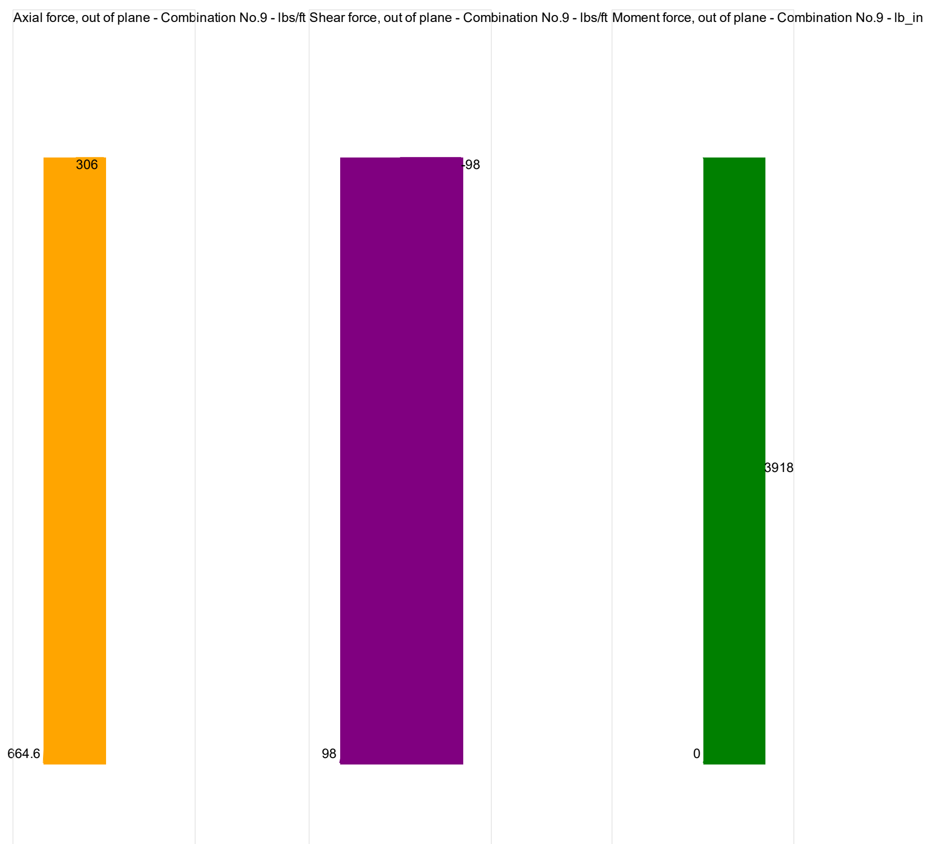
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell6}^2) =$ **840036** in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) =$ **14001** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 6.67 ft

Axial load at max moment loc. of panel P = **485** lb/ft

Compressive stress due to axial load $f_a = P / A =$ **9** psi

Slenderness ratio $(K \times h) / r =$ **61.887** < 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **382.2** psi

$f_a / F_a =$ **0.023**

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **20644** lb/ft

$P / P_a =$ **0.024**

Bending moment at max. moment loc. of panel M = **3918** lb_in/ft

Depth of reinforcement d = **3.813** in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) =$ **40** in

Modular ratio $n = E_s / E_m =$ **16.959**

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m =$ **855** psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m =$ **855** psi

Tension at balance point $T_{bal} = A_s \times F_s =$ **2945** lb/ft

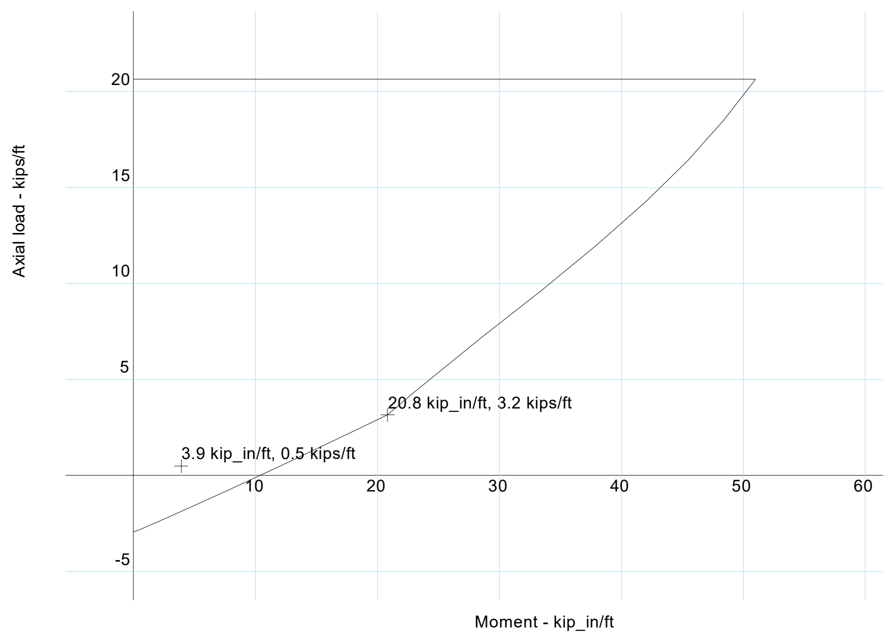
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6099** lb/ft

Axial load at balance point $P_{bal} = C_{bal} - T_{bal} =$ **3154** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3) =$ **20835** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **12005** lb_in/ft

$M / M_c =$ **0.326**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force $V =$ **98** lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) =$ **8.9** in^2/ft

Shear stress $f_v = V / A_{nv} =$ **11** psi

Compressive force $N_v = 0.60 \times P_{DL,t,out} / 1\ ft =$ **306** lb/ft

Moment $M =$ **0** lb_in/ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\ psi)} + 0.25 \times N_v / A$ =**88.6** psi

Allowable shear stress $F_v = \min(F_{vm}, 3 \times \sqrt{(f'_m \times 1\ psi)}) =$ **88.6** psi

Shear utilization $f_v / F_v =$ **0.124**

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.2.07

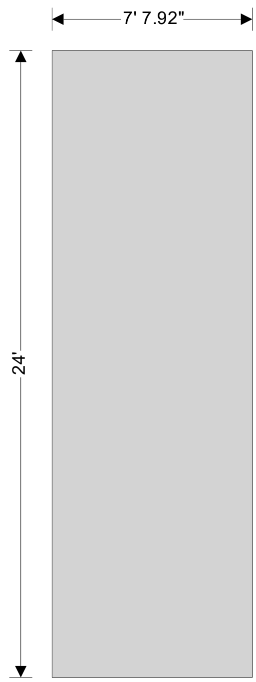
Masonry wall panel details

Bathroom Masonry Walls mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **7.66** ft

Panel height h = **24** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

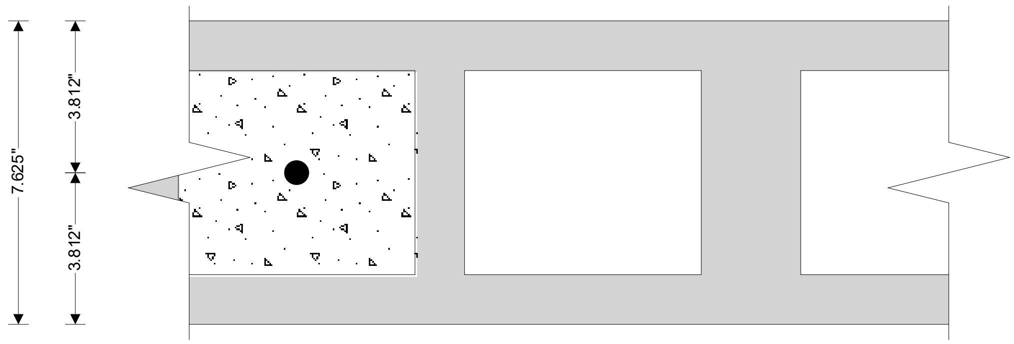
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

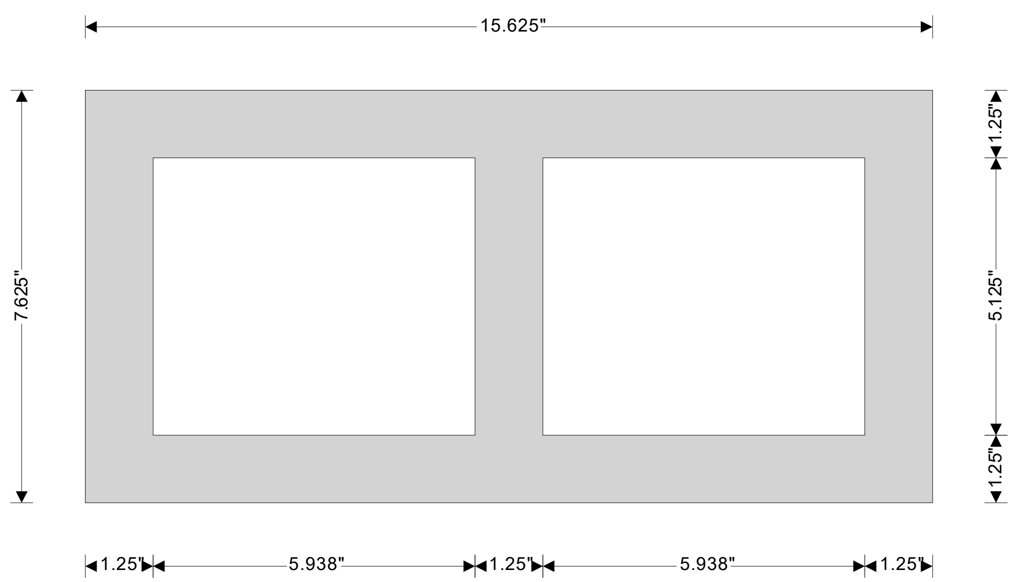
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **1900** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1500** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1350000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **540000** psi

From TMS 402 -11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **51** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.15** in^2/ft

Lateral out-of-plane loads

Wind load on panel W = **22.2** psf

Wind load on parapet W_p = **16.3** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = max(F_p,0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall V_W = **8000** lbs

Vertical loading details

Dead load at supported level DL = **90** lb/ft

Live roof load at supported level LL_r = **60** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.158)

Load combination no.2 DL + LL (0.158)

Load combination no.5 DL + 0.6 × W (0.830)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.618)

Load combination no.8 DL + 0.75 × LL + 0.525 × E_h + 0.525 × E_v + 0.75 × SL (0.532)

Load combination no.9 0.6 × DL + 0.6 × W (0.906)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in^4/ft

Section modulus $S = I / c$ = **98.3** in^3/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

Effective height factor K = **1**

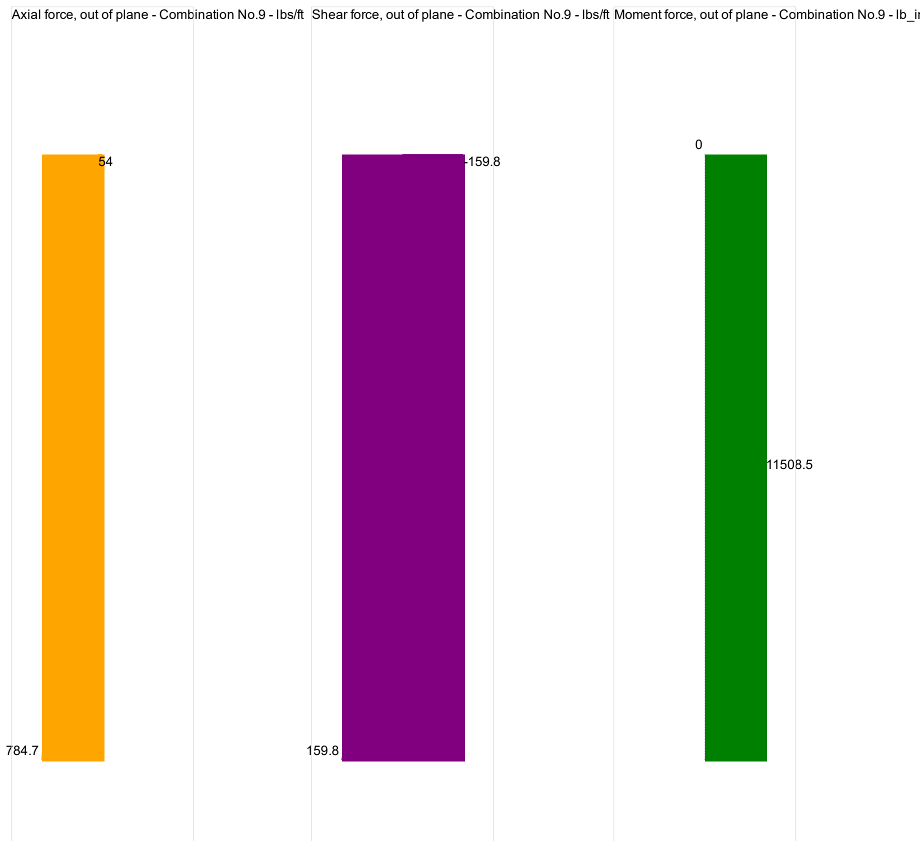
Properties for walls loaded in-plane:

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Bathroom Wall 8				Sheet no./rev. 4	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) = \mathbf{411170}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{8946}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 12 ft

Axial load at max moment loc. of panel $P = \mathbf{419}$ lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{7}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{115.411}$ > 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{138}$ psi

$f_a / F_a = \mathbf{0.051}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{8281}$ lb/ft

$P / P_a = \mathbf{0.051}$

Bending moment at max. moment loc. of panel $M = \mathbf{11508}$ lb_in/ft

Depth of reinforcement $d = \mathbf{3.813}$ in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) = \mathbf{24}$ in

Modular ratio $n = E_s / E_m = \mathbf{21.481}$

Allowable compressive stress due to flexural load $F_b = (0.45) \times f'_m = \mathbf{675}$ psi

Balance point $k_{bal} = n / (F_s / F_b + n) =$ **0.312**

Tensile strain in reinforcement $\varepsilon_s = F_s / E_s =$ **0.001103**

Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**

Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m =$ **675** psi

Tension at balance point $T_{bal} = A_s \times F_s =$ **4909** lb/ft

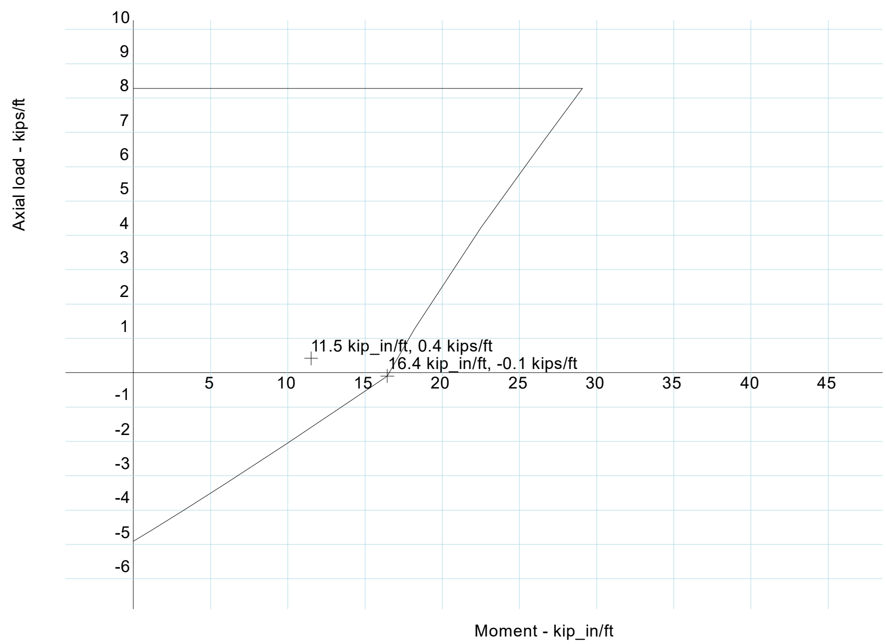
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **4815** lb/ft

Axial load at balance point $P_{bal} = C_{bal} – T_{bal} =$ **-94** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **16448** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **16698** lb_in/ft

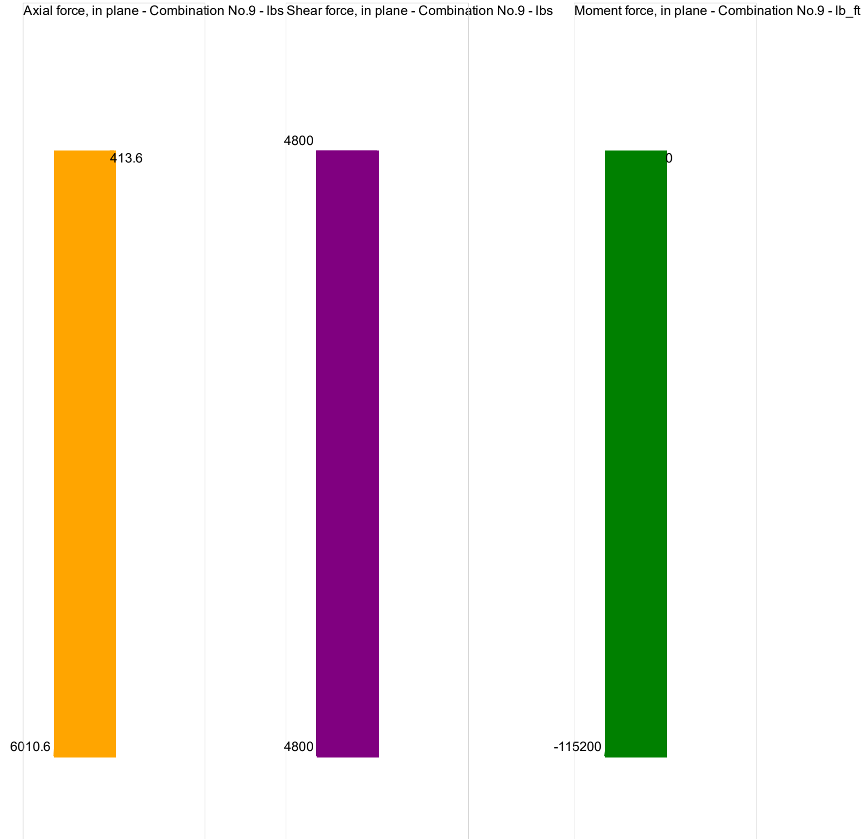
$M / M_c =$ **0.689**



Allowable stress interaction diagram

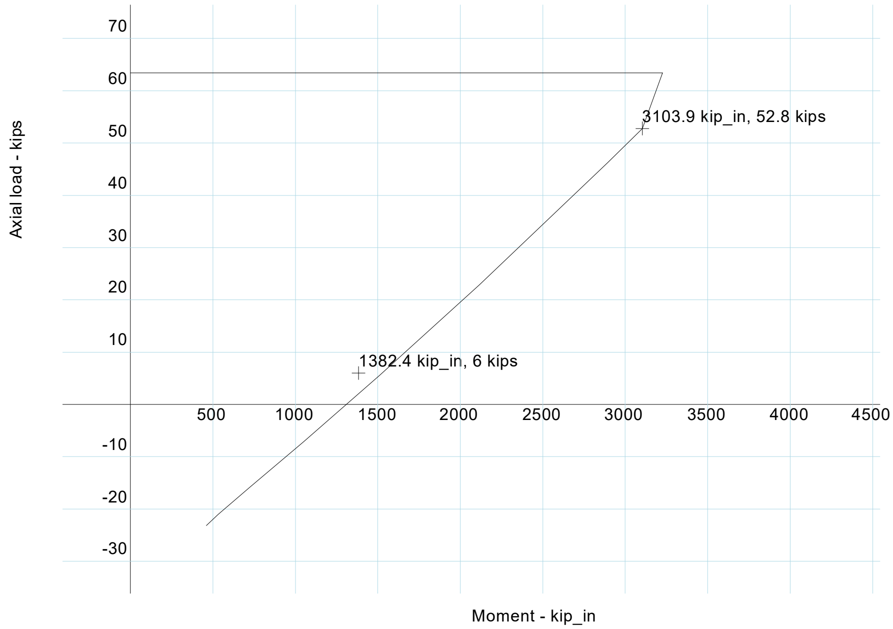
Consider wall at bottom under load combination no.9

Tekla Tedds	Project I-10 Rest Area			Job Ref. B190988.00	
	Section Bathroom Wall 8			Sheet no./rev. 6	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Shear force	$V = \mathbf{4800}$ lbs
Depth to reinforcement	$d_{in} = L - l_b / 4 = \mathbf{7.3}$ ft
Net shear area	$A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf}) = \mathbf{366.9}$ in^2
Shear stress	$f_v = V / A_{nv} = \mathbf{13.1}$ psi
Compressive force	$N_v = 0.6 \times P_{DL,b,in} = \mathbf{6010.6}$ lbs
Moment	$M_v = \mathbf{115200}$ lb_ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L) = \mathbf{46.8}$ psi
Allowable shear stress	$F_v = \min(F_{vm},2 \times \sqrt{(f'_m \times 1 \text{ psi})}) = \mathbf{46.8}$ psi
Shear utilization	$f_v / F_v = \mathbf{0.279}$

Axial load	$P = \mathbf{6011}$ lb
By iteration, tension at balance point	$T_{bal} = \mathbf{17770}$ lb
Compression at balance point	$C_{bal} = \mathbf{70553}$ lb
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal} = \mathbf{52784}$ lb
By iteration, moment at balance point	$M_{bal} = \mathbf{3103930}$ lb_in
Maximum moment from interaction diagram	$M_c = \mathbf{1526007}$ lb_in
	$M_v / M_c = \mathbf{0.906}$



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.2.07

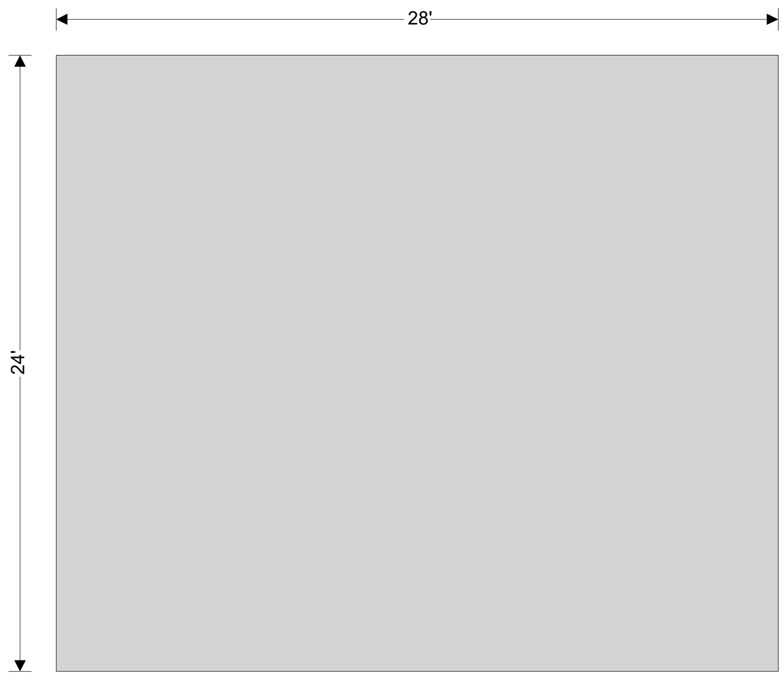
Masonry wall panel details

Bathroom Masonry Walls mid - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **28** ft

Panel height h = **24** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

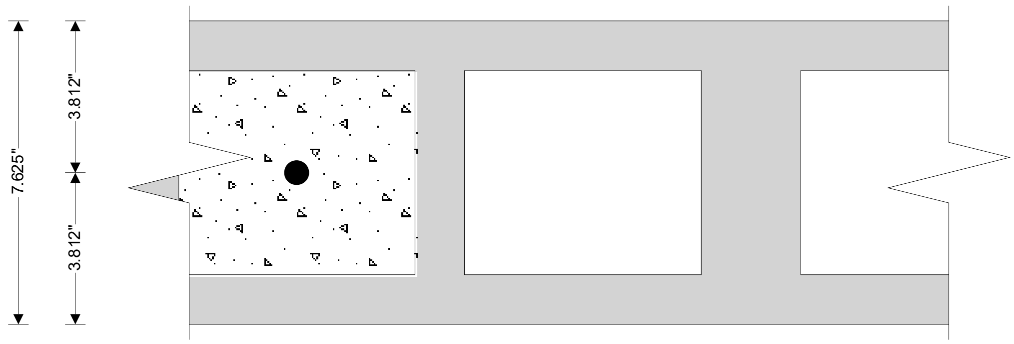
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

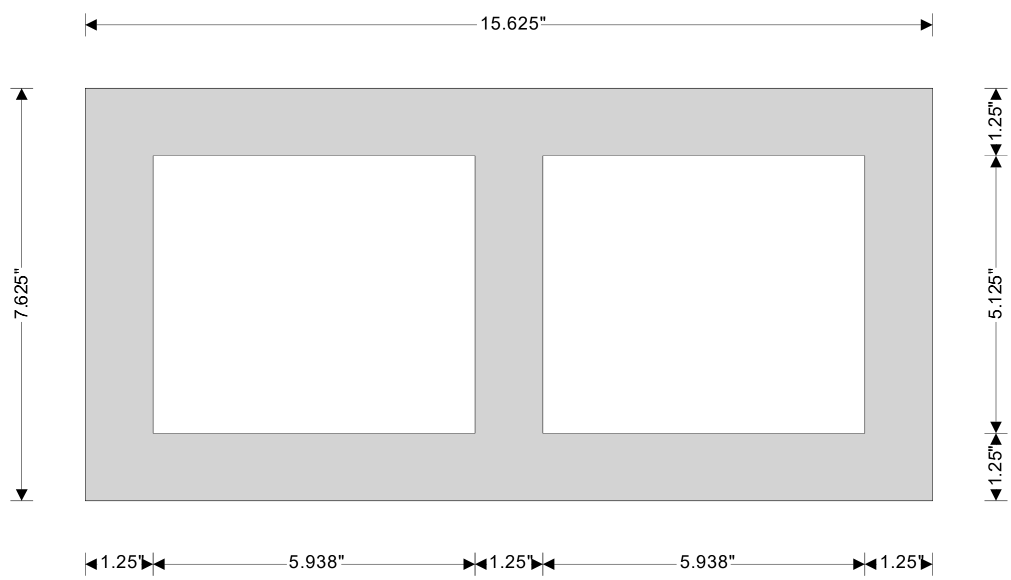
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry	f'_m = **1500** psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m$ = **1350000** psi
Shear modulus of masonry	$G_v = 0.4 \times E_m$ = **540000** psi

From TMS 402 -11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed	F_{t_norm} = **51** psi
Allowable flexural tensile stress parallel to bed	F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi
Vertical reinforcement provided	No.5 bars at 24 in centers
Area of vertical reinforcement	$A_s = \pi \times Dia^2 / (4 \times s)$ = **0.15** in²/ft

Lateral out-of-plane loads

Wind load on panel	W = **22.2** psf
Wind load on parapet	W_p = **16.3** psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**
Seismic load from wall	$E_{wall} = \max(F_p,0.1) \times w_{SW}$ = **20.3** psf
Additional seimic load	E_{add} = **0** psf
Seismic lateral load on panel	$E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall	V_W = **13200** lbs

Vertical loading details

Dead load at supported level	DL = **90** lb/ft
Live roof load at supported level	LL_r = **60** lb/ft
Vertical seismic load factor applied to dead load	$F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1	DL (0.158)
Load combination no.2	DL + LL (0.158)
Load combination no.5	$DL + 0.6 \times W$ (0.670)
Load combination no.7	$DL + 0.75 \times LL + 0.45 \times W + 0.75 \times (LL_r$ or SL or RL) (0.500)
Load combination no.8	$DL + 0.75 \times LL + 0.525 \times E_h + 0.525 \times E_v + 0.75 \times SL$ (0.532)
Load combination no.9	$0.6 \times DL + 0.6 \times W$ (0.689)

Properties of masonry section

Cross-sectional area	$A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in²/ft
Properties for walls loaded out-of-plane:	
Moment of inertia	$I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in⁴/ft
Section modulus	$S = I / c$ = **98.3** in³/ft
Radius of gyration	$r = \sqrt{[I / A]}$ = **2.495** in
Effective height factor	K = **1**
Properties for walls loaded in-plane:	

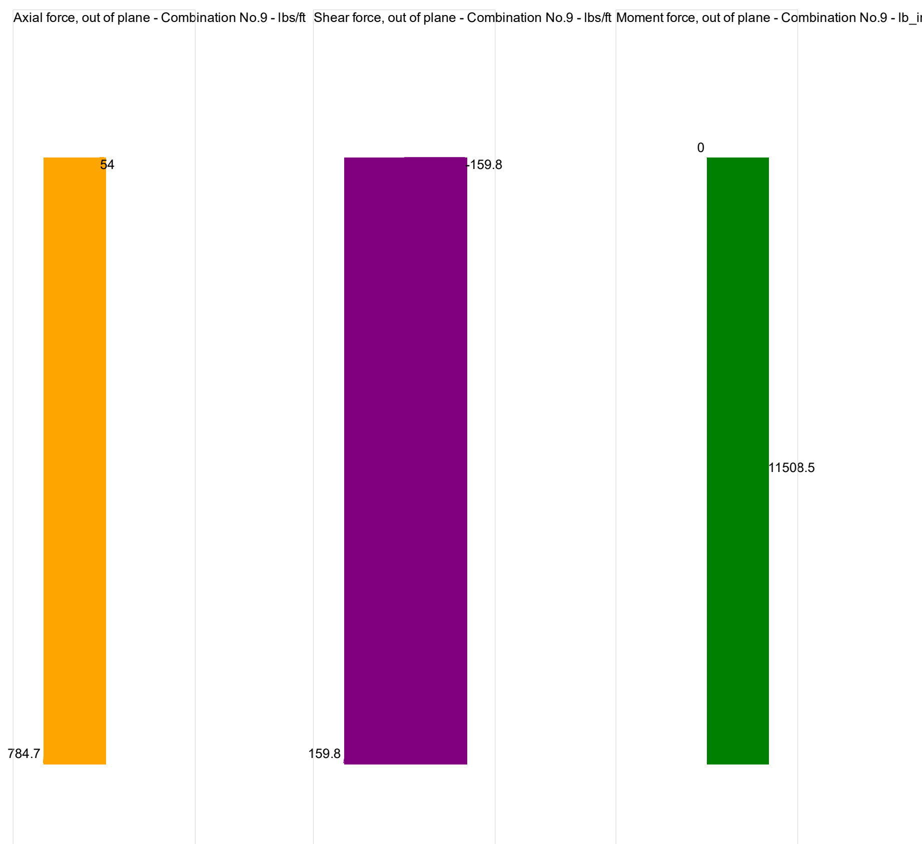
Net moment of inertia

$I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) =$ **16112148** in^4

Net section modulus

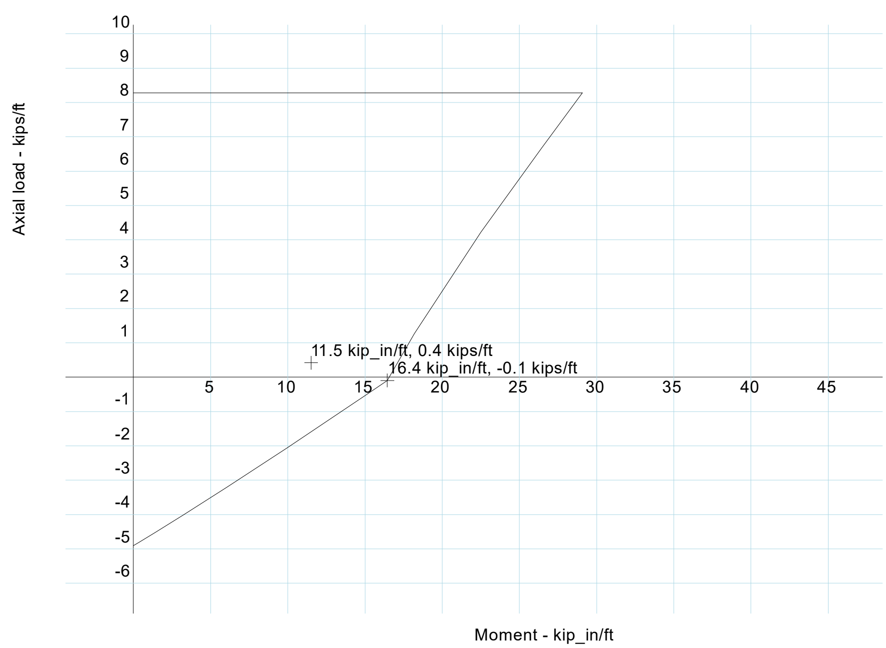
$S_{x\_net} = I_{x\_net} / (L / 2) =$ **95906** in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location	12 ft
Axial load at max moment loc. of panel	P = **419** lb/ft
Compressive stress due to axial load	$f_a = P / A =$ **7** psi
Slenderness ratio	$(K \times h) / r =$ **115.411** > 99
Allowable compressive stress due to axial load	$F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 =$ **138** psi
	$f_a / F_a =$ **0.051**
Allowable compressive force	$P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 =$ **8281** lb/ft
	$P / P_a =$ **0.051**
Bending moment at max. moment loc. of panel	M = **11508** lb_in/ft

Depth of reinforcement	d = **3.813** in
Effective width per bar	$b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in})$ = **24** in
Modular ratio	$n = E_s / E_m$ = **21.481**
Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m$ = **675** psi
Balance point	$k_{bal} = n / (F_s / F_b + n)$ = **0.312**
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s$ = **0.001103**
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal})$ = **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m$ = **675** psi
Tension at balance point	$T_{bal} = A_s \times F_s$ = **4909** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2$ = **4815** lb/ft
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal}$ = **-94** lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3)$ = **16448** lb_in/ft
Maximum moment from interaction diagram	M_c = **16698** lb_in/ft
	M / M_c = **0.689**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force	V = **159.8** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **14.9** in^2/ft
Shear stress	$f_v = V / A_{nv}$ = **10.7** psi
Compressive force	$N_v = 0.60 \times P_{DL,t,out} / 1\text{ ft}$ = **54** lb/ft
Moment	M = **0** lb_in/ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_v / A$ =**77.7** psi
Allowable shear stress	$F_v = \min(F_{vm}, 3 \times \sqrt{(f'_m \times 1\text{ psi})})$ = **77.7** psi
Shear utilization	f_v / F_v = **0.138**

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

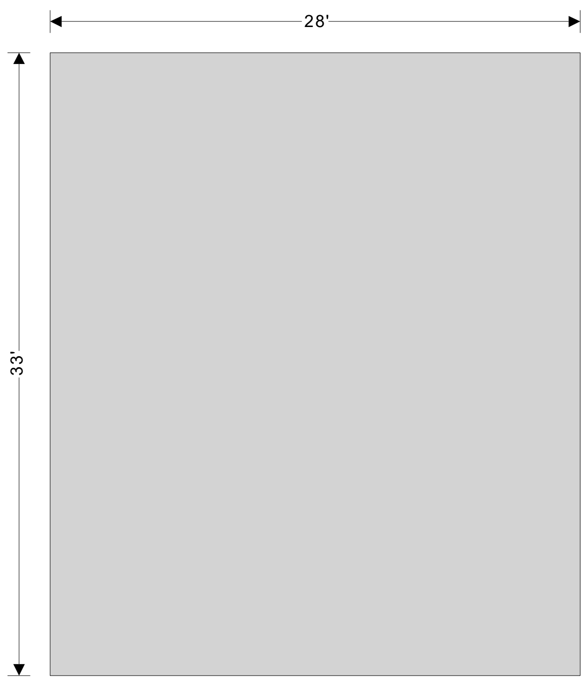
Masonry wall panel details

Shear wall 10 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **28** ft

Panel height h = **33** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

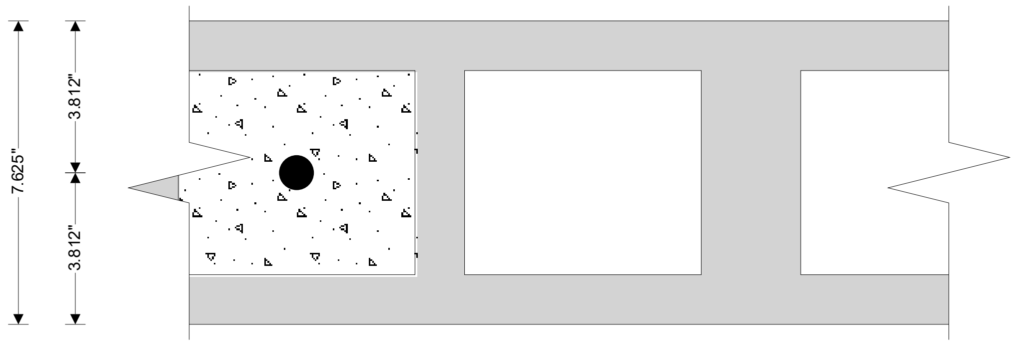
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

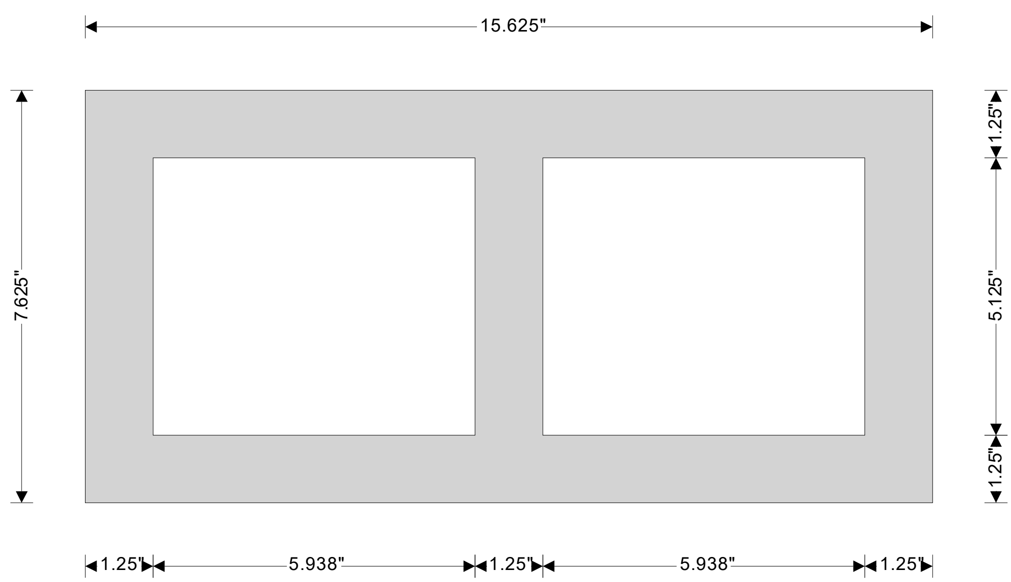
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **1900** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry — f'_m = **1900** psi

Modulus of elasticity for masonry — $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry — $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed — F_{t_norm} = **44** psi

Allowable flexural tensile stress parallel to bed — F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement — f_y = **60000** psi

Allowable tensile stress in reinforcement — F_s = **32000** psi

Modulus of elasticity for reinforcement — E_s = **29000000** psi

Vertical reinforcement provided — No.7 bars at 24 in centers

Area of vertical reinforcement — $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.30** in²/ft

Lateral out-of-plane loads

Wind load on panel — W = **16.3** psf

Wind load on parapet — W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) — $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall — $E_{wall} = \max(F_p,0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load — E_{add} = **0** psf

Seismic lateral load on panel — $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall — V_W = **27100** lbs

Vertical loading details

Vertical seismic load factor applied to dead load — $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 — DL (0.303)

Load combination no.2 — DL + LL (0.303)

Load combination no.3 — DL + (LL_r or SL or RL) (0.303)

Load combination no.4 — DL + 0.75 × LL + 0.75 × (LL_r or SL or RL) (0.303)

Load combination no.5 — DL + 0.6 × W (0.627)

Load combination no.7 — DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.470)

Load combination no.9 — 0.6 × DL + 0.6 × W (0.637)

Properties of masonry section

Cross-sectional area — $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia — $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in⁴/ft

Section modulus — $S = I / c$ = **98.3** in³/ft

Radius of gyration — $r = \sqrt{[I / A]}$ = **2.495** in

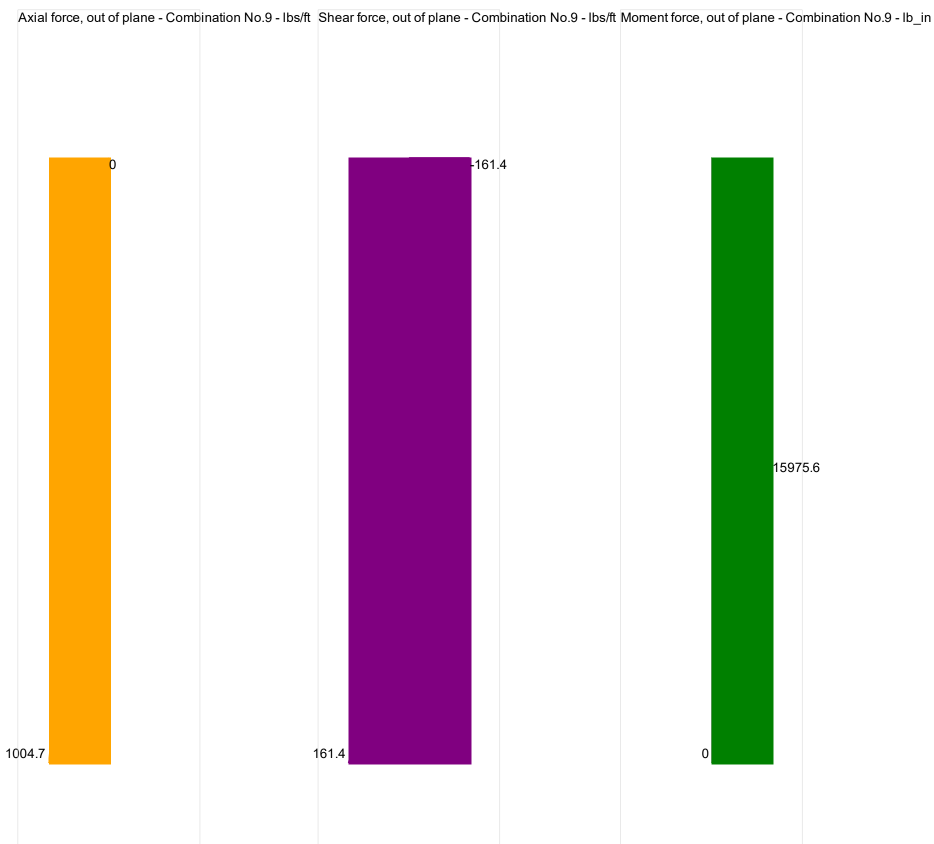
Effective height factor — K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) = \mathbf{16112148}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{95906}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 16.5 ft

Axial load at max moment loc. of panel P = **502** lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{8.3}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{158.690} > 99$

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{92.4}$ psi

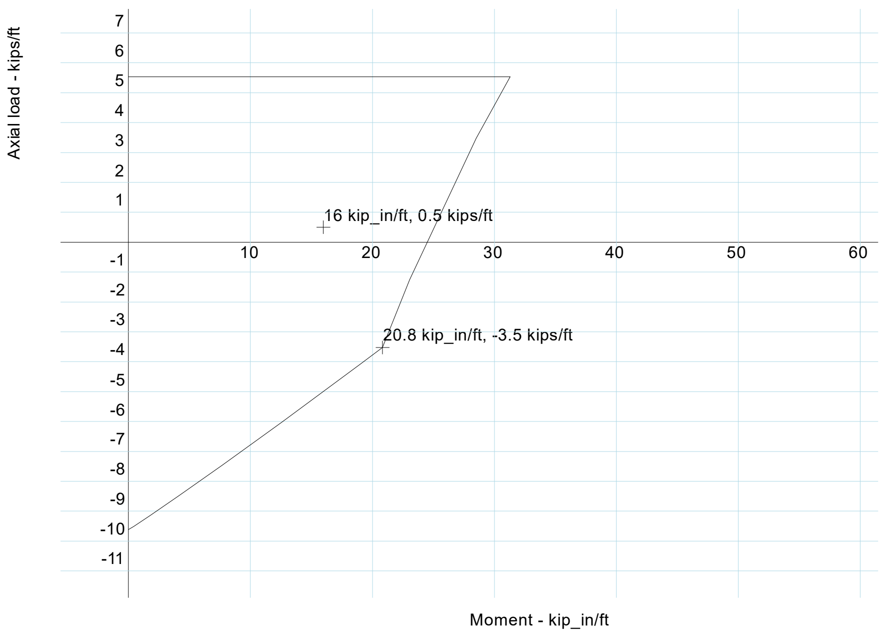
$f_a / F_a = \mathbf{0.090}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{5535}$ lb/ft

$P / P_a = \mathbf{0.091}$

Bending moment at max. moment loc. of panel M = **15976** lb_in/ft

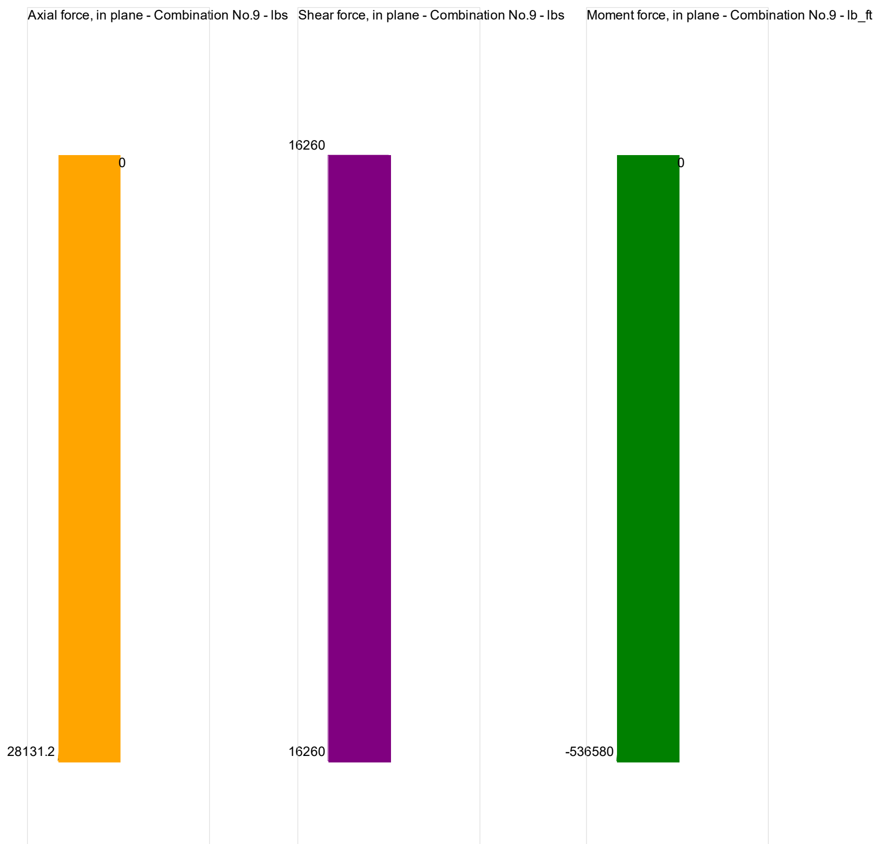
Depth of reinforcement	d = **3.813** in
Effective width per bar	b_{eff} = min(s, 6 × t_{nom}, 72 in) = **24** in
Modular ratio	n = E_s / E_m = **16.959**
Allowable compressive stress due to flexural load	F_b = (0.45) × f'_m = **855** psi
Balance point	k_{bal} = n / (F_s / F_b + n) = **0.312**
Tensile strain in reinforcement	ε_s = F_s / E_s = **0.001103**
Compressive strain in masonry	ε_m = ε_s × k_{bal} / (1 - k_{bal}) = **0.000500**
Compressive stress at balance point	f_{bal} = ε_m × E_m = **855** psi
Tension at balance point	T_{bal} = A_s × F_s = **9621** lb/ft
Compression at balance point	C_{bal} = k_{bal} × d × f_{bal} / 2 = **6099** lb/ft
Axial load at balance point	P_{bal} = C_{bal} – T_{bal} = **-3522** lb/ft
Moment at balance point	M_{bal} = T_{bal} × (d - t / 2) + C_{bal} × (t / 2 – k_{bal} × d / 3) = **20835** lb_in/ft
Maximum moment from interaction diagram	M_c = **25083** lb_in/ft
	M / M_c = **0.637**



Allowable stress interaction diagram

Consider wall at bottom under load combination no.9

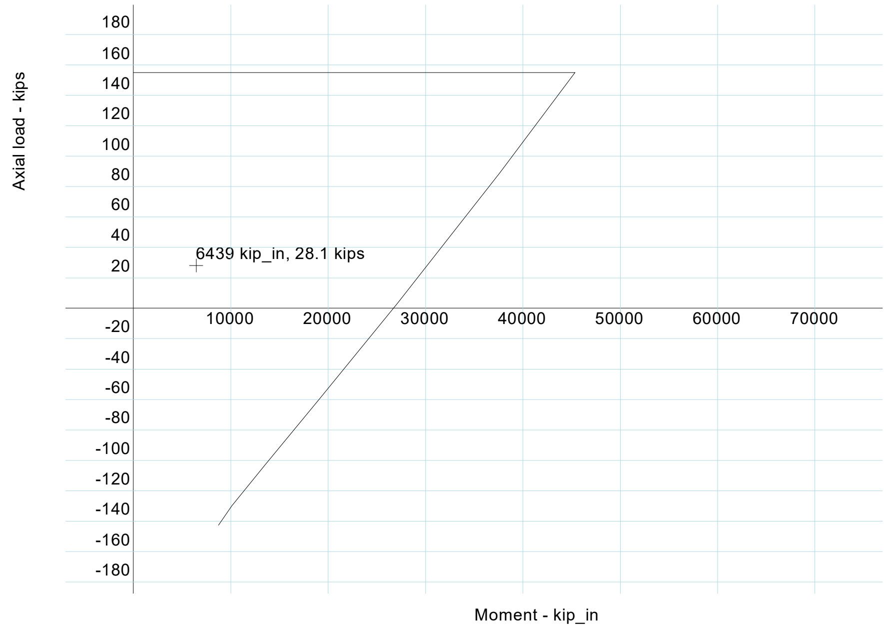
Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Shear wall 10, full				Sheet no./rev. 6	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Shear force	$V = \mathbf{16260}$ lbs
Depth to reinforcement	$d_{in} = L - l_b / 4 = \mathbf{27.7}$ ft
Net shear area	$A_{nv} = 2 \times d_{in} \times t_{bf} + d_{in} / s_{grout} \times (l_b / (N_{web} + 1)) \times (t - 2 \times t_{bf}) = \mathbf{1384.3}$ in^2
Shear stress	$f_v = V / A_{nv} = \mathbf{11.7}$ psi
Compressive force	$N_v = 0.6 \times P_{DL,b,in} = \mathbf{28131.2}$ lbs
Moment	$M_v = \mathbf{536580}$ lb_ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M_v / (V \times d_{in})),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / (A \times L) = \mathbf{53.2}$ psi
Allowable shear stress	$F_v = \min(F_{vm},2 \times \sqrt{(f'_m \times 1 \text{ psi})}) \times 0.75 = \mathbf{39.9}$ psi
Shear utilization	$f_v / F_v = \mathbf{0.294}$

Axial load	$P = \mathbf{28131}$ lb
By iteration, tension at balance point	$T_{bal} = \mathbf{101464}$ lb
Compression at balance point	$C_{bal} = \mathbf{337465}$ lb
Axial load at balance point	$P_{bal} = C_{bal} - T_{bal} = \mathbf{236002}$ lb
By iteration, moment at balance point	$M_{bal} = \mathbf{54732749}$ lb_in
Maximum moment from interaction diagram	$M_c = \mathbf{30149137}$ lb_in
	$M_v / M_c = \mathbf{0.214}$

Tekla. Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Shear wall 10, full				Sheet no./rev. 7	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Allowable stress interaction diagram

MASONRY WALL PANEL DESIGN TO MSJC-13

Using the allowable stress design method

Tedds calculation version 2.2.07

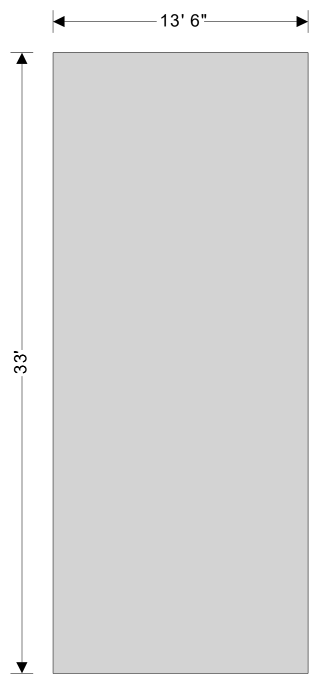
Masonry wall panel details

Shear wall 10 - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **13.5** ft

Panel height h = **33** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

Seismic wall classification Nonparticipating

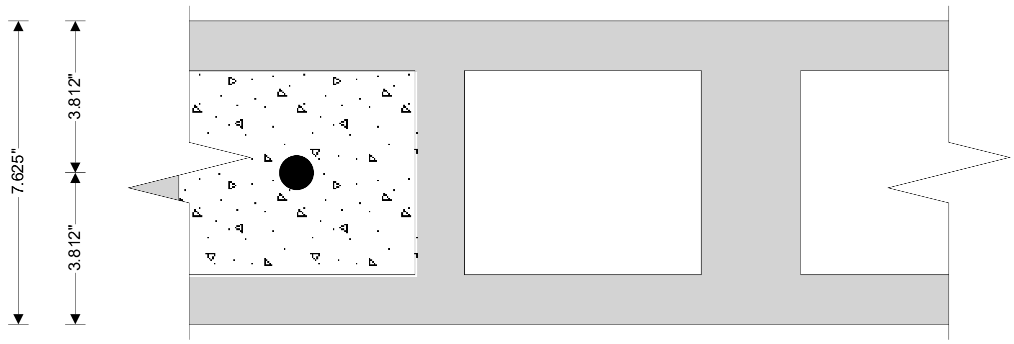
No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in

Tekla Tedds	Project: I-10 Rest Area	Job Ref.: B190988.00
	Section: Shear wall 10, half	Sheet no./rev.: 2
	Calc. by: ALM · Date: 9/3/2020 · Chk'd by: NJA · Date: 9/3/2020	App'd by: · Date:



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

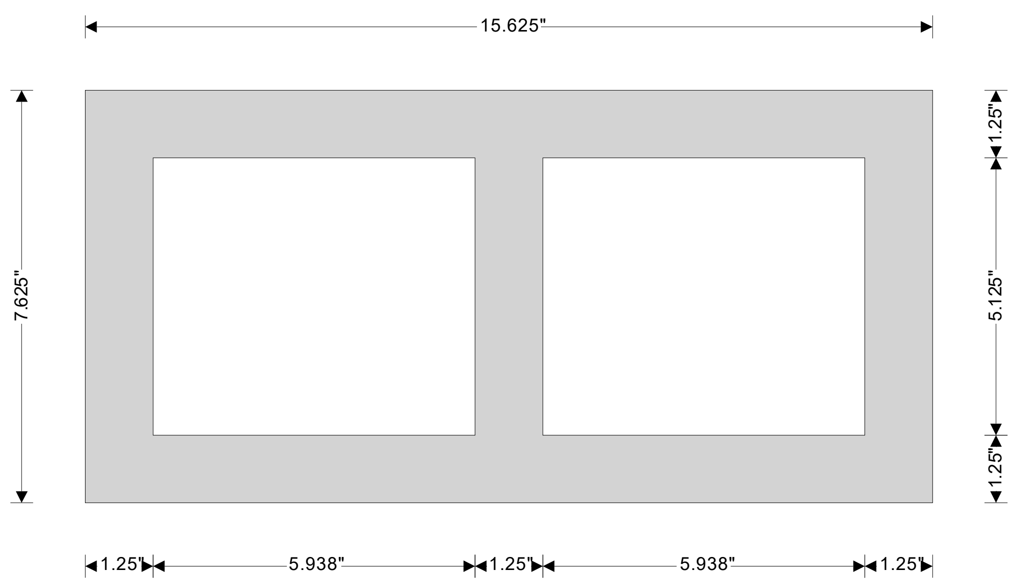
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-13 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1900** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -13 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **44** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.7 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.30** in²/ft

Lateral out-of-plane loads

Wind load on panel W = **16.3** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Wind shear load on wall V_W = **14** lbs

Vertical loading details

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.303)

Load combination no.2 DL + LL (0.303)

Load combination no.3 DL + (LL_r or SL or RL) (0.303)

Load combination no.4 DL + 0.75 × LL + 0.75 × (LL_r or SL or RL) (0.303)

Load combination no.5 DL + 0.6 × W (0.627)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.470)

Load combination no.9 0.6 × DL + 0.6 × W (0.637)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in²/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in⁴/ft

Section modulus $S = I / c$ = **98.3** in³/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

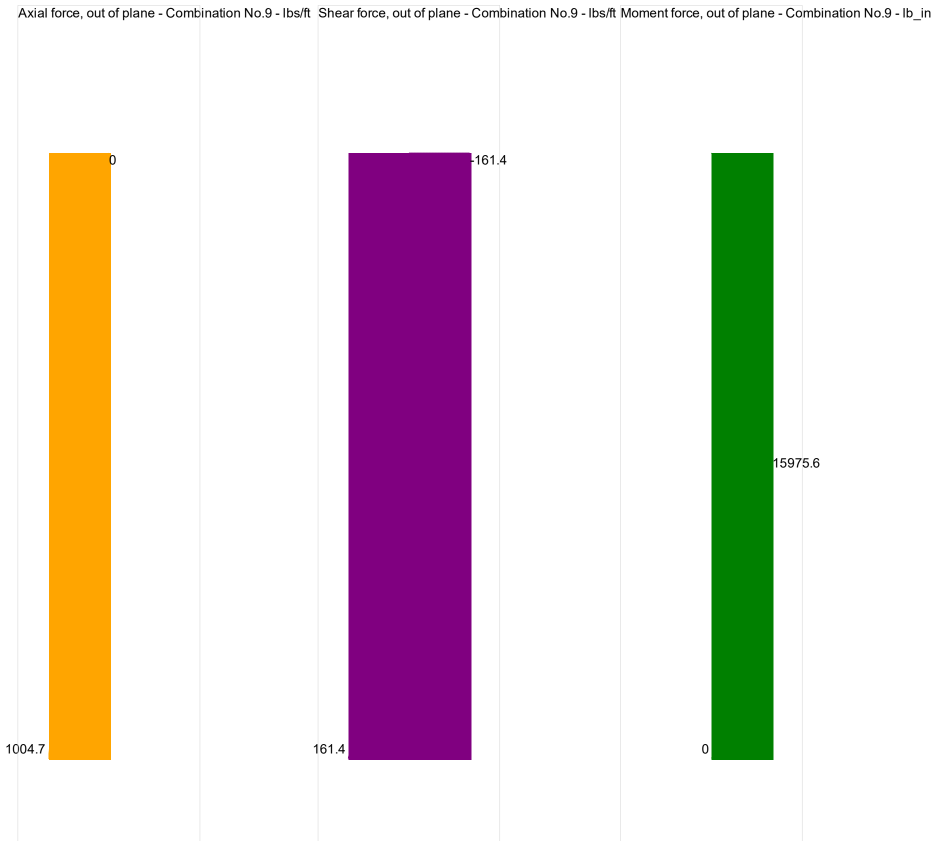
Effective height factor K = **1**

Properties for walls loaded in-plane:

Net moment of inertia $I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) = \mathbf{2081114}$ in^4

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{25693}$ in^3

Consider wall at maximum moment location under load combination no.9



Maximum moment location 16.5 ft

Axial load at max moment loc. of panel $P = \mathbf{502}$ lb/ft

Compressive stress due to axial load $f_a = P / A = \mathbf{8.3}$ psi

Slenderness ratio $(K \times h) / r = \mathbf{158.690} > 99$

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times (70 \times r / (K \times h))^2 = \mathbf{92.4}$ psi

$f_a / F_a = \mathbf{0.090}$

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times (70 \times r / (K \times h))^2 = \mathbf{5535}$ lb/ft

$P / P_a = \mathbf{0.091}$

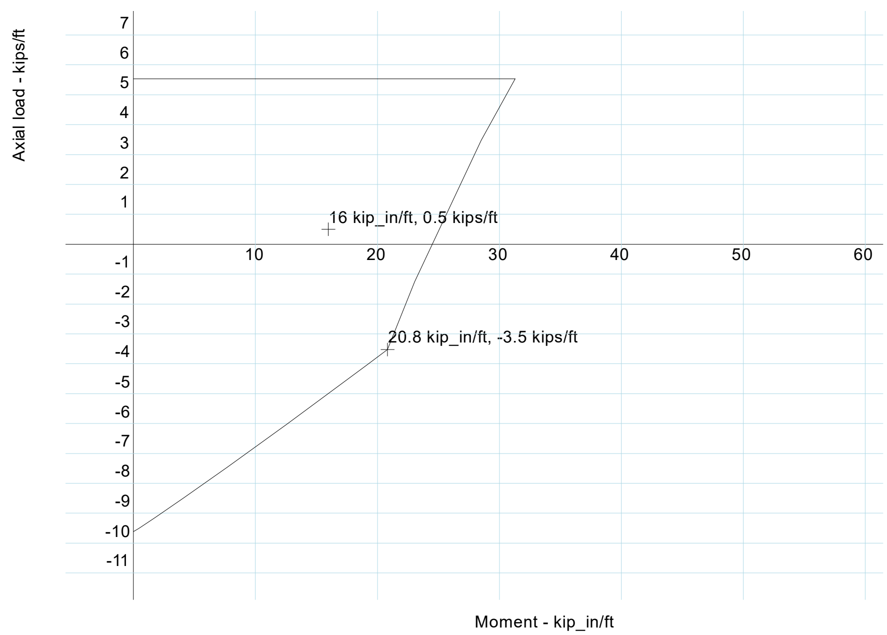
Bending moment at max. moment loc. of panel $M = \mathbf{15976}$ lb_in/ft

Depth of reinforcement $d = \mathbf{3.813}$ in

Effective width per bar $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{24}$ in

Modular ratio $n = E_s / E_m = \mathbf{16.959}$

Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m =$ **855** psi
Balance point	$k_{bal} = n / (F_s / F_b + n) =$ **0.312**
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s =$ **0.001103**
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m =$ **855** psi
Tension at balance point	$T_{bal} = A_s \times F_s =$ **9621** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6099** lb/ft
Axial load at balance point	$P_{bal} = C_{bal} – T_{bal} =$ **-3522** lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **20835** lb_in/ft
Maximum moment from interaction diagram	$M_c =$ **25083** lb_in/ft
	$M / M_c =$ **0.637**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force	$V =$ **161.4** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) =$ **14.9** in²/ft
Shear stress	$f_v = V / A_{nv} =$ **10.8** psi
Compressive force	$N_v = 0.60 \times P_{DL,t,out} / 1\ ft =$ **0** lb/ft
Moment	$M =$ **0** lb_in/ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\ psi)} + 0.25 \times N_v / A$ =**87.2** psi
Allowable shear stress	$F_v = min(F_{vm},3 \times \sqrt{(f'_m \times 1\ psi)}) =$ **87.2** psi
Shear utilization	$f_v / F_v =$ **0.124**

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.2.07

Masonry wall panel details

Interior masonry wall (typ) - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **30** ft

Panel height h = **13.33** ft

Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

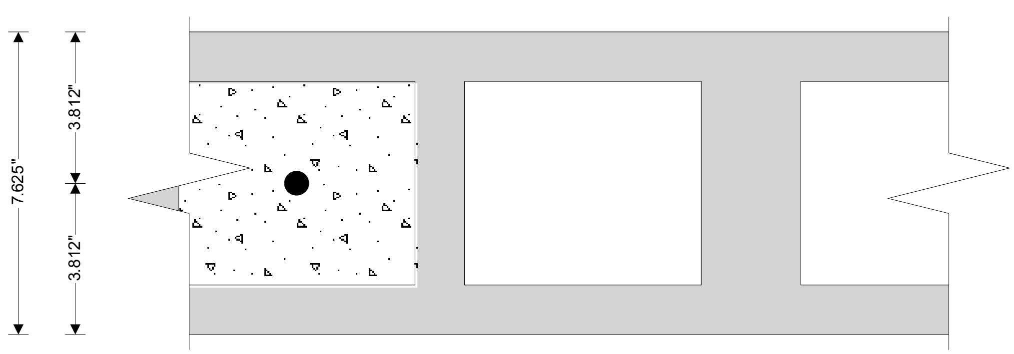
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

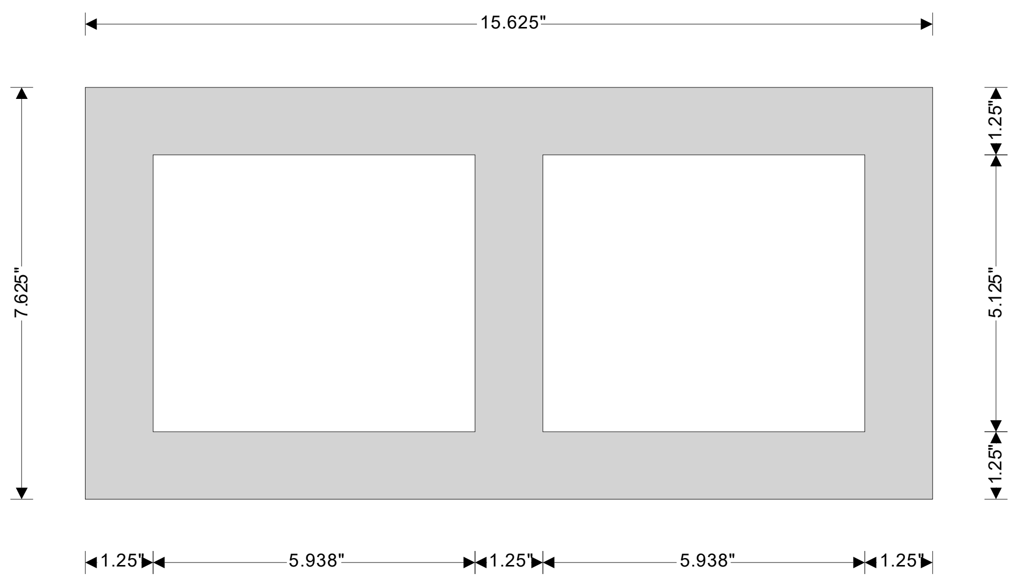
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **2800** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry	f'_m = **2000** psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m$ = **1800000** psi
Shear modulus of masonry	$G_v = 0.4 \times E_m$ = **720000** psi

From TMS 402 -11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed	F_{t_norm} = **42** psi
Allowable flexural tensile stress parallel to bed	F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi

Vertical reinforcement provided — No.5 bars at 48 in centers

Area of vertical reinforcement — $A_s = \pi \times Dia^2 / (4 \times s) = \mathbf{0.08}$ in^2/ft

Lateral out-of-plane loads

Wind load on panel — $W = \mathbf{5}$ psf

Wind load on parapet — $W_p = \mathbf{5}$ psf

Seismic load factor (ASCE7 12.11.1) — $F_p = 0.4 \times S_{DS} \times I_e = \mathbf{0.4}$

Seismic load from wall — $E_{wall} = \max(F_p, 0.1) \times w_{SW} = \mathbf{17.4}$ psf

Additional seimic load — $E_{add} = \mathbf{0}$ psf

Seismic lateral load on panel — $E = E_{wall} + E_{add} = \mathbf{17.4}$ psf

Lateral in-plane loads

Vertical loading details

Vertical seismic load factor applied to dead load — $F_{Ev} = 0.2 \times S_{DS} = \mathbf{0.2}$

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 — DL (0.027)

Load combination no.2 — DL + LL (0.027)

Load combination no.5 — DL + 0.6 × W (0.083)

Load combination no.9 — 0.6 × DL + 0.6 × W (0.086)

Properties of masonry section

Cross-sectional area — $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = \mathbf{52.7}$ in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia — $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = \mathbf{358.4}$ in^4/ft

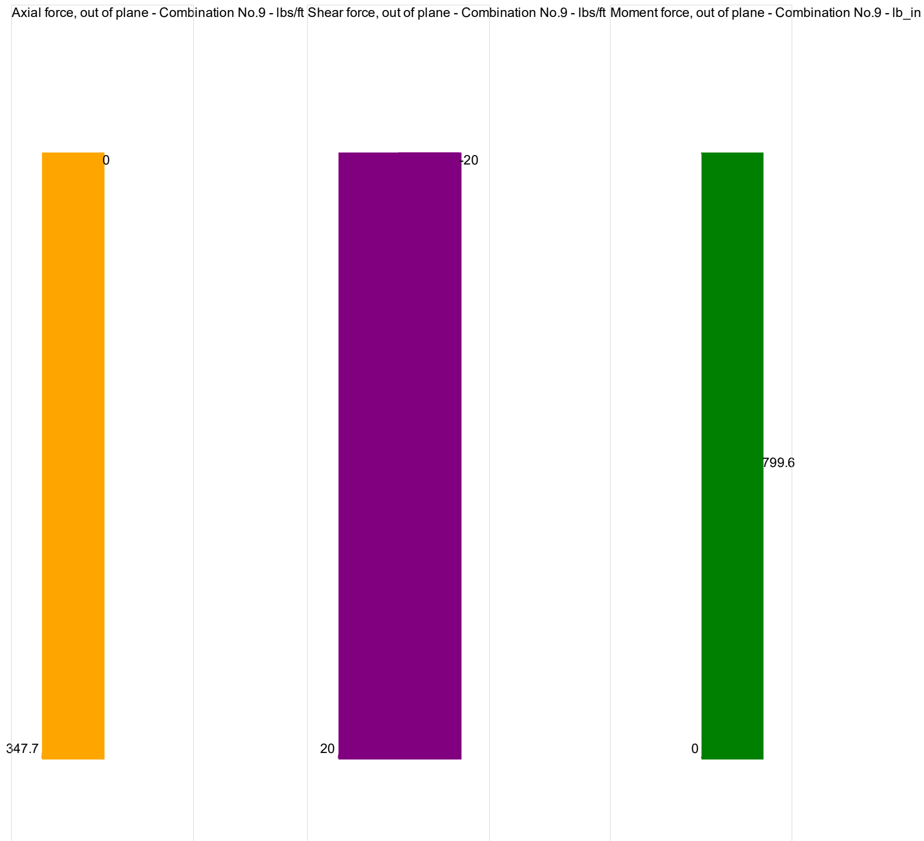
Section modulus — $S = I / c = \mathbf{94}$ in^3/ft

Radius of gyration — $r = \sqrt{[I / A]} = \mathbf{2.608}$ in

Effective height factor — $K = \mathbf{1}$

Consider wall at maximum moment location under load combination no.9

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Interior Masonry Wall				Sheet no./rev. 4	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date



Maximum moment location	6.67 ft
Axial load at max moment loc. of panel	P = **174** lb/ft
Compressive stress due to axial load	$f_a = P / A =$ **3.3** psi
Slenderness ratio	$(K \times h) / r =$ **61.341** < 99
Allowable compressive stress due to axial load	$F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **404** psi
	$f_a / F_a =$ **0.008**
Allowable compressive force	$P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **21263** lb/ft
	$P / P_a =$ **0.008**
Bending moment at max. moment loc. of panel	M = **800** lb_in/ft
Depth of reinforcement	d = **3.813** in
Effective width per bar	$b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) =$ **48** in
Modular ratio	$n = E_s / E_m =$ **16.111**
Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m =$ **900** psi
Balance point	$k_{bal} = n / (F_s / F_b + n) =$ **0.312**
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s =$ **0.001103**
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) =$ **0.000500**
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m =$ **900** psi
Tension at balance point	$T_{bal} = A_s \times F_s =$ **2454** lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 =$ **6420** lb/ft

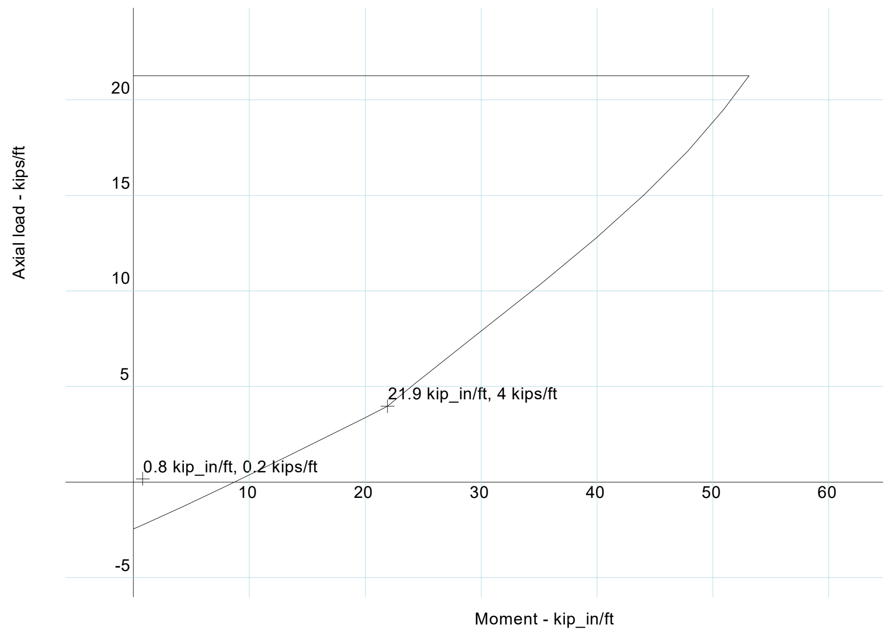
Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Interior Masonry Wall				Sheet no./rev. 5	
	Calc. by ALM	Date 9/3/2020	Chk'd by NJA	Date 9/3/2020	App'd by	Date

Axial load at balance point $P_{bal} = C_{bal} – T_{bal} =$ **3965** lb/ft

Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) =$ **21931** lb_in/ft

Maximum moment from interaction diagram $M_c =$ **9295** lb_in/ft

$M / M_c =$ **0.086**



Allowable stress interaction diagram

Consider wall at top under load combination no.9

Shear force $V =$ **20** lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) =$ **7.4** in^2/ft

Shear stress $f_v = V / A_{nv} =$ **2.7** psi

Compressive force $N_v = 0.60 \times P_{DL,t,out} / 1\text{ ft} =$ **0** lb/ft

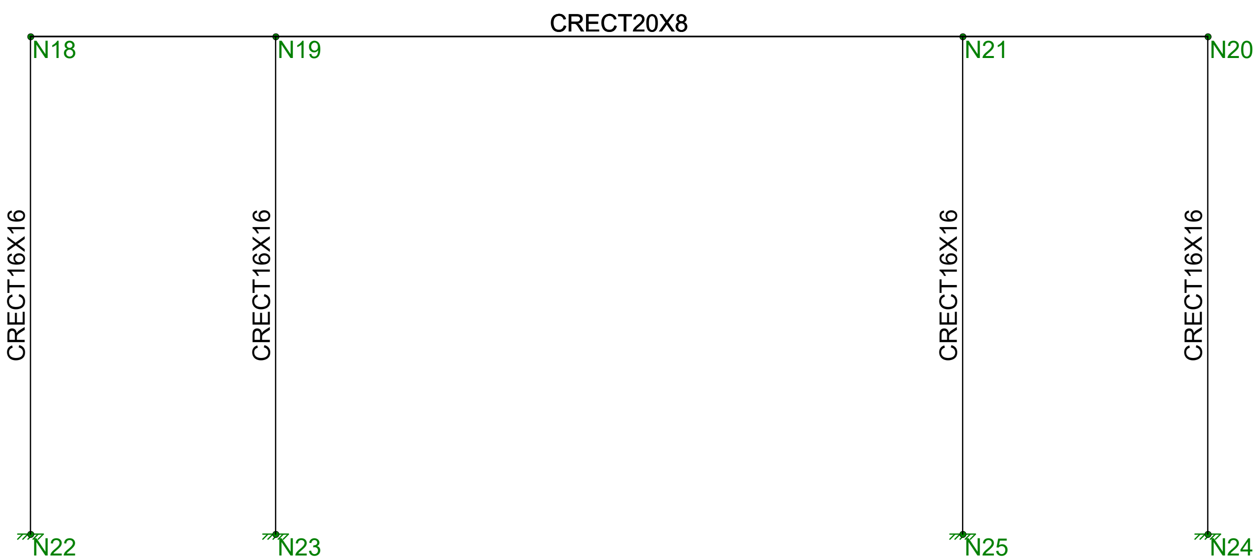
Moment $M =$ **0** lb_in/ft

Allowable masonry shear stress $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_v / A$ =**89.4** psi

Allowable shear stress $F_v = \min(F_{vm},3 \times \sqrt{(f'_m \times 1\text{ psi})}) =$ **89.4** psi

Shear utilization $f_v / F_v =$ **0.030**

Entrance Way at Front of Building Moment Frame



GAI Consultants	I-10 Moment Frame	SK - 1
NJA		July 7, 2020 at 11:16 AM
B190988.00		Moment Frame with LRFD.R3D

Basic Load Cases

	BLC Description	Category	X Gr...	Y Gra...	Z Gra...	Joint	Point	Distrib...	Area(Member)	Surface(Plate/Wall)
1	Self	DL		-1						
2	Truss Vert	DL				4				
3	Beam Dead	DL				2				
4	Beam Wind	WL				2				
5	Truss Hor	WL				4				

Load Combinations

	Description	S...	P...	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	BLC	Fa...
1	ASD down		Y		DL	1																		
2	Wind		Y				WL	1																
3	ASD max	Yes	Y		DL	1	WL	.6																
4	ASD min	Yes	Y		DL	.6	WL	.6																
5	LRFD max	Yes	Y		DL	1.2	WL	1																
6	LRFD min	Yes	Y		DL	.9	WL	1																

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N22	max	-9.311	4	-12.512	3	0	6	0	6	0	6	157.465	5
2		min	-15.604	5	-22.691	6	0	3	0	3	0	3	94.326	4
3	N23	max	-10.042	3	42.854	5	0	6	0	6	0	6	167.109	6
4		min	-16.924	6	23.604	4	0	3	0	3	0	3	99.471	3
5	N24	max	-9.218	4	27.799	5	0	6	0	6	0	6	157.645	5
6		min	-15.33	5	16.361	4	0	3	0	3	0	3	94.416	4
7	N25	max	-10.644	4	4.34	3	0	6	0	6	0	6	171.017	5
8		min	-17.83	5	-6.199	6	0	3	0	3	0	3	102.221	4
9	Totals:	max	-39.349	4	47.703	5	0	6						
10		min	-65.582	5	23.851	4	0	3						

Envelope Concrete Beam Design Results

	Member	Shape	UC Ma...	Loc[ft]	UC LC	UC Ma...	Loc[ft]	UC LC	Shear ...	Loc[ft]	UC LC	Phi*Mnz ...	Phi*Mnz ...	Phi*Vny[k]
1	M16	CRECT2...	.948	7.767	5	.883	33.656	6	.494	6.472	5	87.868	87.868	52.123

Envelope Concrete Column Design Results

	Column	Shape	UC M...	Loc[ft]	UC LC	Shear...	LC	Loc[ft]	Dir	Phi used	Pn[k]	Mny[k-ft]	Mnz[k-ft]	Vny[k]	Vnz[k]
1	M21	CRECT...	.999	0	6	.389	5	0	y	.9	-25.225		174.922	53.014	53.014
2	M22	CRECT...	.907	0	6	.285	6	0	y	.9	46.092		204.741	80.336	80.336
3	M23	CRECT...	.952	0	6	.444	5	0	y	.9	-7.238		198.873	53.505	53.505
4	M24	CRECT...	.875	0	6	.257	5	0	y	.9	34.358		200.062	80.336	80.336

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318

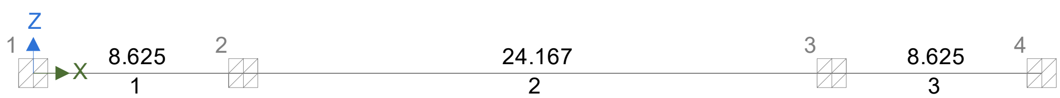
Tedds calculation version 3.3.03

ANALYSIS

Tedds calculation version 1.0.31

Geometry

Geometry (ft) - Concrete (4000 150)

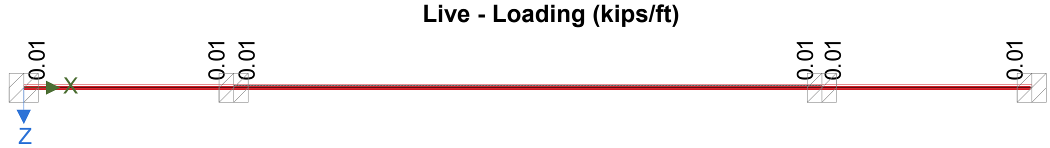


Span	Length (ft)	Section	Start Support	End Support
1	8.63	R 8x20	Fixed	Fixed
2	24.17	R 8x20	Fixed	Fixed
3	8.63	R 8x20	Fixed	Fixed
R 12x20: Area 240 in^2, Inertia Major 8000 in^4, Inertia Minor 2880 in^4, Shear area parallel to Minor 200 in^2, Shear area parallel to Major 200 in^2				
R 8x20: Area 160 in^2, Inertia Major 5333 in^4, Inertia Minor 853 in^4, Shear area parallel to Minor 133 in^2, Shear area parallel to Major 133 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

Loading

Self weight included

Live - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Live	UDL	GlobalZ	0.01 kips/ft

Tekla Tedds GAI Consultants, Inc. 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA 16066	Project I-10 Rest Area				Job Ref. B190988.00	
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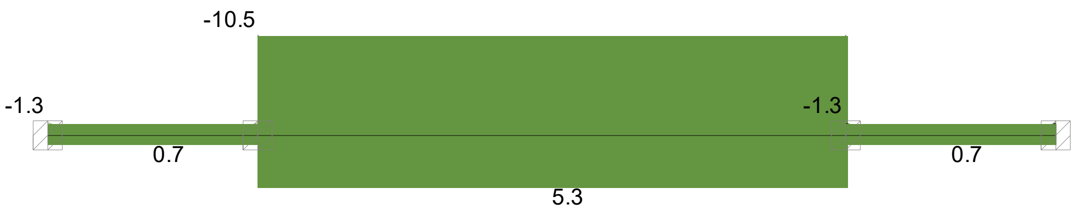
	Calc. by NJA	Date 9/4/2020	Chk'd by	Date	App'd by	Date

Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	E = (w_c / 1 lb/ft^3)$^{1.5}$ × 33 psi × (f'_c / 1 psi)$^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **1.5** in
Cover to side reinforcement	c_{nom_s} = **1.5** in

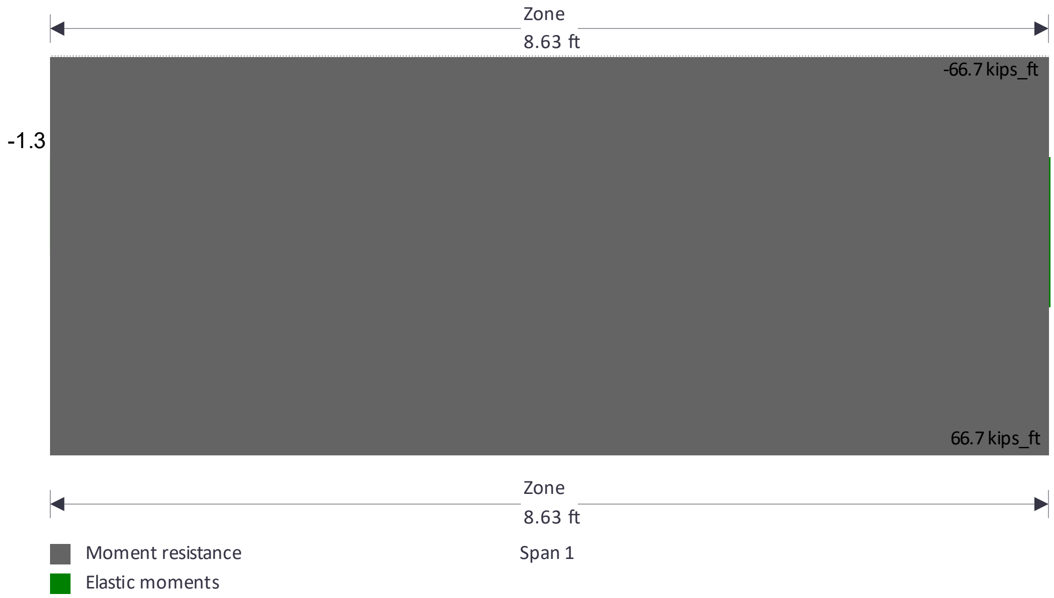
Beam - Span 1

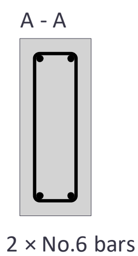
Rectangular section details

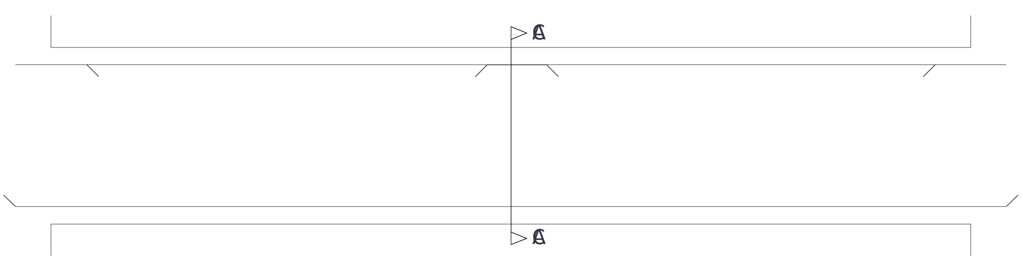
Section width	b = **8** in
Section depth	h = **20** in

Beam Deepened per Architectural design.
Sided reinforcement added for deeper
beam per ACI314-14 Section 9.7.2.3.

Moment design







Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z2\_max\_red})$ = **0.670** kip_ft

Effective depth to tension reinforcement d = **17.75** in

Tension reinforcement provided 2 × No.6 bars

Area of tension reinforcement provided $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.473** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in

Depth to neutral axis $c = a / \beta_1$ = **2.293** in

Net tensile strain in extreme tension fibers — $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = \mathbf{0.02022}$

T T T T T T

Strength reduction factor (cl.21.2.1) — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.90}$

Nominal moment strength — $M_n = A_{s,prov} \times f_y \times (d - a / 2) = \mathbf{74.112}$ kip_ft

Design moment strength — $\phi M_n = M_n \times \phi_f = \mathbf{66.701}$ kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1} = \mathbf{3.500}$ in

Service load stress in reinforcement (cl.24.3.2) — $f_s = 2/3 \times f_y = \mathbf{40000}$ psi

Clear cover of reinforcement — $c_c = c_{nom\_b} + \phi_v = \mathbf{1.875}$ in

Maximum allowable bot bar spacing (Table 24.3.2) — $s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) = \mathbf{10.313}$ in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor — $K_w = \mathbf{1.00}$

Reinforcement yield strength factor — $K_f = 0.4 + f_y / 100000 \text{ psi} = \mathbf{1.00}$

Minimum thickness of beam (Table 9.3.1.1) — $h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f = \mathbf{5.595}$ in

TT T T T T T

<u>Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)</u>

Factored bending moment at section — $M_u = \text{abs}(M_{m1\_s1\_z2\_min\_red}) = \mathbf{1.339}$ kip_ft

Effective depth to tension reinforcement — $d = \mathbf{17.75}$ in

Tension reinforcement provided — 2 × No.6 bars

Area of tension reinforcement provided — $A_{s,prov} = \mathbf{0.884}$ in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min} = \max(3 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}, 200 \text{ psi}) \times b \times d / f_y = \mathbf{0.473}$ in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$

Depth of equivalent rectangular stress block — $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) = \mathbf{1.949}$ in

Depth to neutral axis — $c = a / \beta_1 = \mathbf{2.293}$ in

Net tensile strain in extreme tension fibers — $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = \mathbf{0.02022}$

T T T T T T

Strength reduction factor (cl.21.2.1) — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.90}$

Nominal moment strength — $M_n = A_{s,prov} \times f_y \times (d - a / 2) = \mathbf{74.112}$ kip_ft

Design moment strength — $\phi M_n = M_n \times \phi_f = \mathbf{66.701}$ kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{t,max} = s_{top} + \phi_{m1\_s1\_z2\_t\_L1} = \mathbf{3.500}$ in

Service load stress in reinforcement (cl.24.3.2) — $f_s = 2/3 \times f_y = \mathbf{40000}$ psi

Clear cover of reinforcement — $c_c = c_{nom\_t} + \phi_v = \mathbf{1.875}$ in

Maximum allowable top bar spacing (Table 24.3.2) — $s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) = \mathbf{10.313}$ in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor — $K_w = \mathbf{1.00}$

Reinforcement yield strength factor — $K_f = 0.4 + f_y / 100000 \text{ psi} = \mathbf{1.00}$

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f$ = **5.595** in

TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1)$ = **2.750** in

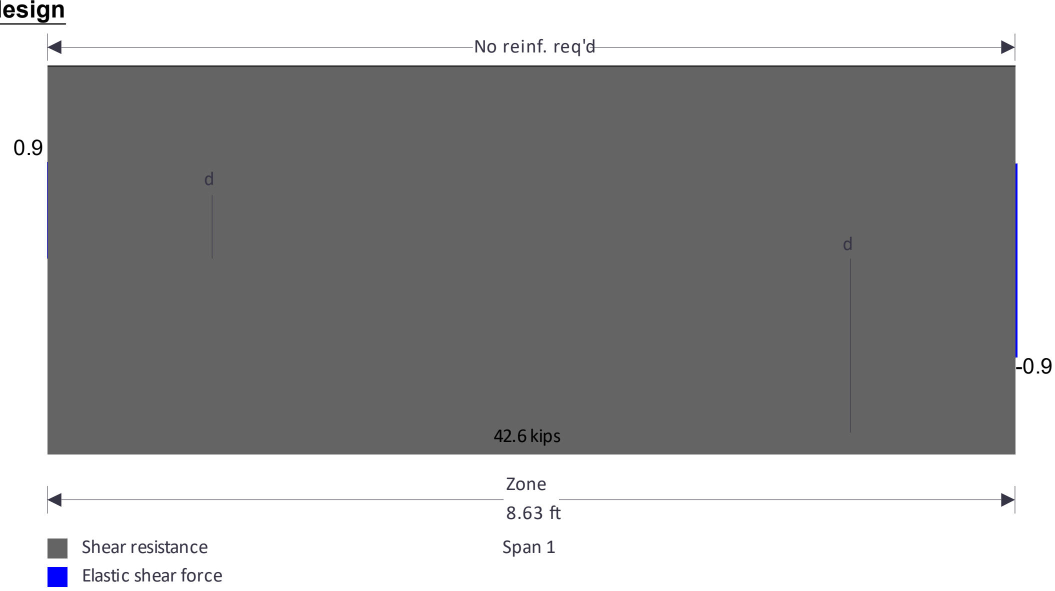
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

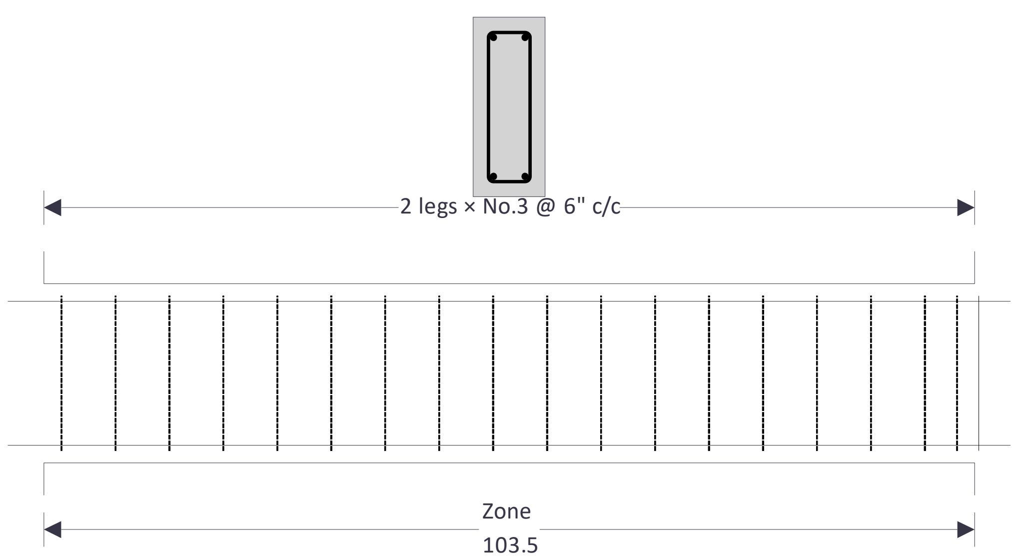
Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1})) / (N_{m1\_s1\_z2\_b\_L1} - 1)$ = **2.750** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

TT T T T T T

Shear design





Rectangular section in shear

Concrete weight modification factor — λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) — N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less ϕV_c / 2) — Between 0.00 ft and 8.63 ft

Maximum reinforcement shear strength — $\phi V_{s,max} = \phi_s \times 8\ psi \times \sqrt{(\min(f'_c, 10000psi) / 1\ psi)} \times b \times d$ = **53.506** kips

Minimum area of shear reinf. (Table 9.6.3.3) — $A_{sv,min} = \max(50\ psi, 0.75\ psi \times \sqrt{(f'_c / 1\ psi)}) \times b / \min(f_y, 60000\ psi)$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone — d = **17.625** in

Concrete shear strength (eqn. 22.5.5.1) — $\phi V_c = \phi_s \times \lambda \times 2\ psi \times \sqrt{(f'_c / 1\ psi)} \times b \times d$ = **13.376** kips

Design shear force within zone — V_u = **1.678** kips

Reinf. shear strength required (eqn. 22.5.1.1) — $\phi V_s = \max(V_u - \phi V_c, 0\ kips)$ = **0.000** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) — $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d)$ = **0.000** in^2/ft

Area of shear reinforcement required — $A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft

Shear reinforcement provided — 2 legs × No.3 @ 6" c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T Tϕ T T

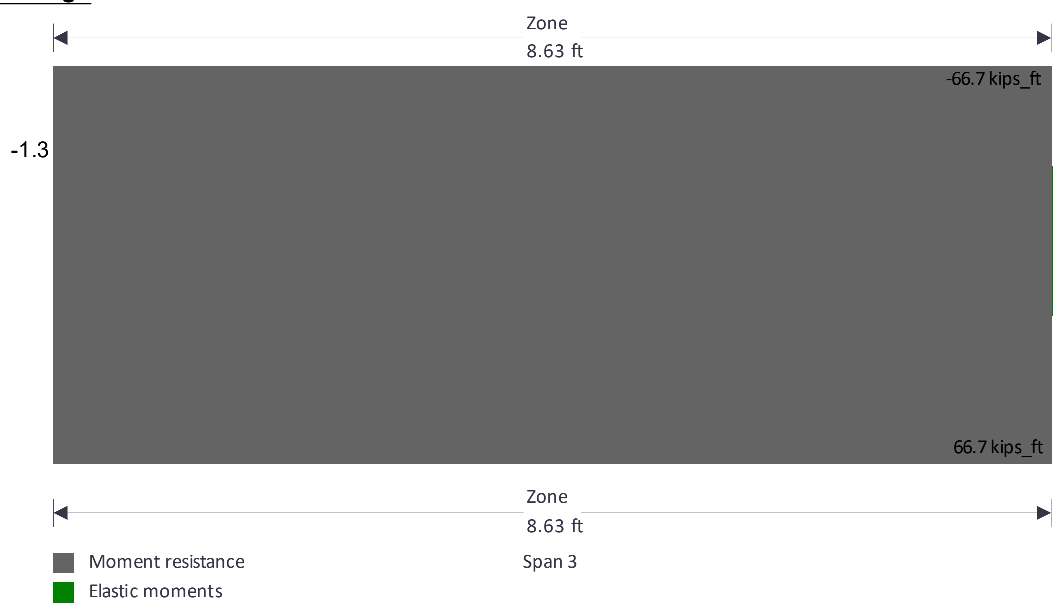
Beam - Span 3

Rectangular section details

Section width — b = **8** in

Section depth — h = **20** in

Moment design



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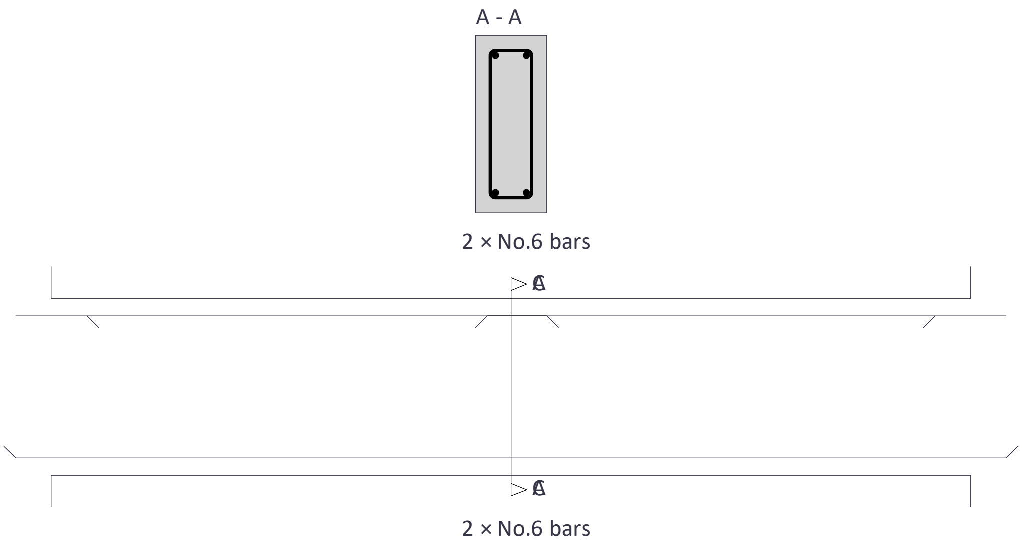
	Section Center Area Moment Frame Top Beam			Sheet no./rev. 7	
	Calc. by NJA	Date 9/4/2020	Chk'd by	Date	App'd by	Date



Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — $M_u = abs(M_{m1\_s3\_z2\_max\_red})$ = **0.670** kip_ft

Effective depth to tension reinforcement — d = **17.75** in

Tension reinforcement provided — 2 × No.6 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **0.884** in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **0.473** in^2

TT T T T T T T T T T T T

Stress block depth factor (cl.22.2.2.4.3) — $\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block — $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **1.949** in

Depth to neutral axis — $c = a / \beta_1$ = **2.293** in

Net tensile strain in extreme tension fibers — $\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.02022**

T T T T T T

Strength reduction factor (cl.21.2.1) — $\phi_f = min(max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**

Nominal moment strength — $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **74.112** kip_ft

Design moment strength — $\phi M_n = M_n \times \phi_f$ = **66.701** kip_ft

TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{b,max} = s_{bot} + \phi_{m1\_s3\_z2\_b\_L1}$ = **3.500** in

Service load stress in reinforcement (cl.24.3.2) — $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement — $c_c = c_{nom\_b} + \phi_v$ = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) — $s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

TT T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor — K_w = **1.00**

Reinforcement yield strength factor — $K_f = 0.4 + f_y / 100000\ psi$ = **1.00**

Minimum thickness of beam (Table 9.3.1.1) — $h_{min} = (L_{m1\_s3} / 18.5) \times K_w \times K_f$ = **5.595** in

TT T T T T T

Zone 1 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s3\_z2\_min\_red}$) = **1.339** kip_ft
Effective depth to tension reinforcement	d = **17.75** in
Tension reinforcement provided	2 × No.6 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **0.884** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'c / 1 psi), 200 psi) × b × d / f_y = **0.473** in^2
TT T T	T T T T T T T T T
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'c × b) = **1.949** in
Depth to neutral axis	c = a / β_1 = **2.293** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.02022**
	T T T T T T
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **74.112** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **66.701** kip_ft
	TT T T T T T T T T

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{t,max}$ = s_{top} + $\phi_{m1\_s3\_z2\_t\_L1}$ = **3.500** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_t} + ϕ_v = **1.875** in
Maximum allowable top bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in
TT	T T T T T T T

Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	K_f = 0.4 + f_y / 100000 psi = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	h_{min} = (L_{m1_s3} / 18.5) × K_w × K_f = **5.595** in
	TT T T T T T

Spacing limits for reinforcement

Top bar clear spacing	s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s3\_z2\_v}$) + $\phi_{m1\_s3\_z2\_t\_L1}$ × $N_{m1\_s3\_z2\_t\_L1}$)) / ($N_{m1\_s3\_z2\_t\_L1}$ - 1) = **2.750** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s3\_z2\_v}$) + $\phi_{m1\_s3\_z2\_b\_L1}$ × $N_{m1\_s3\_z2\_b\_L1}$)) / ($N_{m1\_s3\_z2\_b\_L1}$ - 1) = **2.750** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	TT T T T T T

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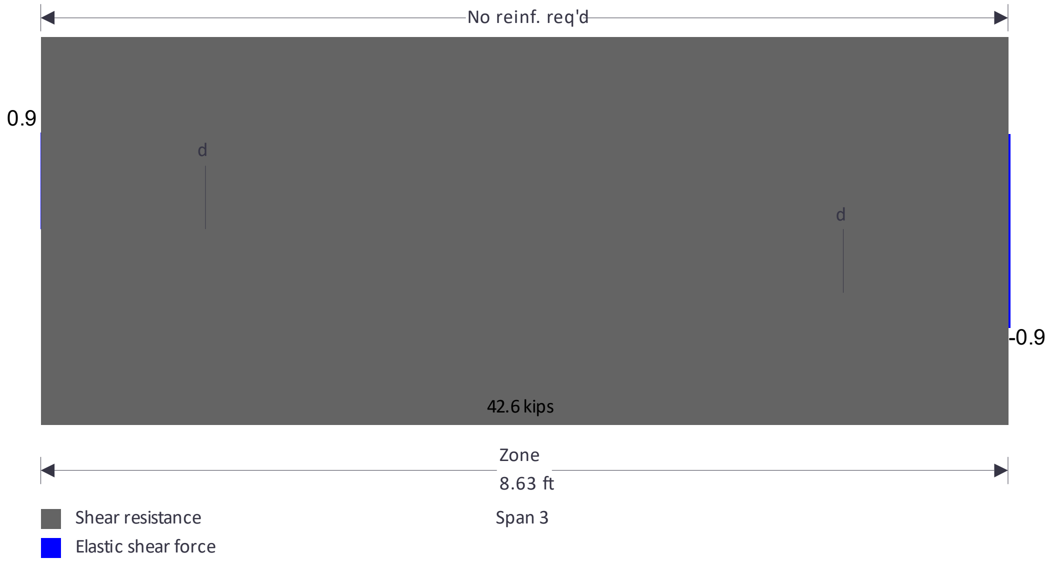
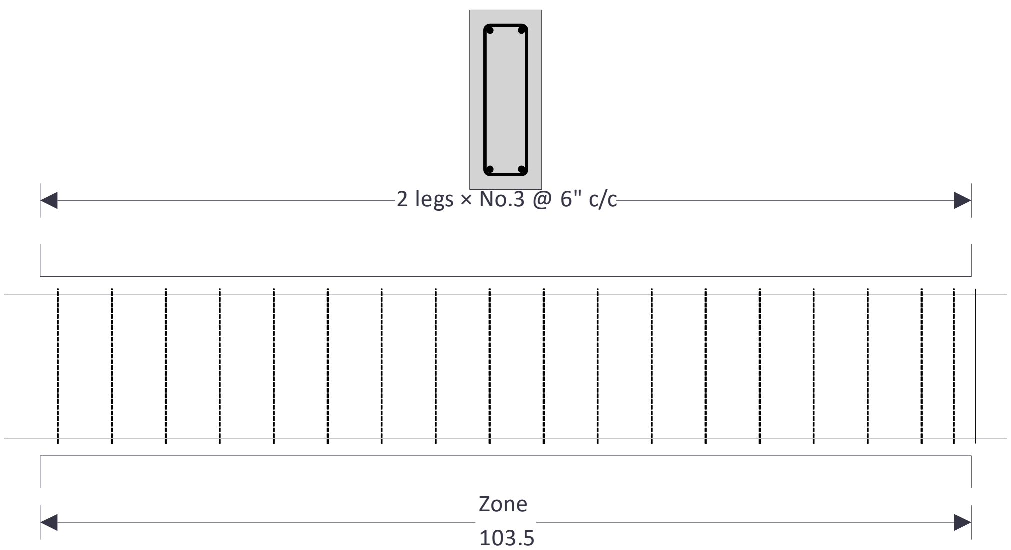
	Section Center Area Moment Frame Top Beam			Sheet no./rev. 9	
	Calc. by NJA	Date 9/4/2020	Chk'd by	Date	App'd by	Date

Shear design





Rectangular section in shear

Concrete weight modification factor λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less ϕV_c / 2) Between 0.00 ft and 8.63 ft

Maximum reinforcement shear strength $\phi V_{s,max} = \phi_s \times 8\text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **53.506** kips

Minimum area of shear reinf. (Table 9.6.3.3) $A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.080** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone d = **17.625** in

Concrete shear strength (eqn. 22.5.5.1) $\phi V_c = \phi_s \times \lambda \times 2\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})} \times b \times d$ = **13.376** kips

Design shear force within zone V_u = **0.614** kips

Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **0.000** kips

Area of design shear reinf. req'd (eqn. 22.5.10.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.000** in^2/ft

Area of shear reinforcement required $A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft

Shear reinforcement provided 2 legs × No.3 @ 6" c/c

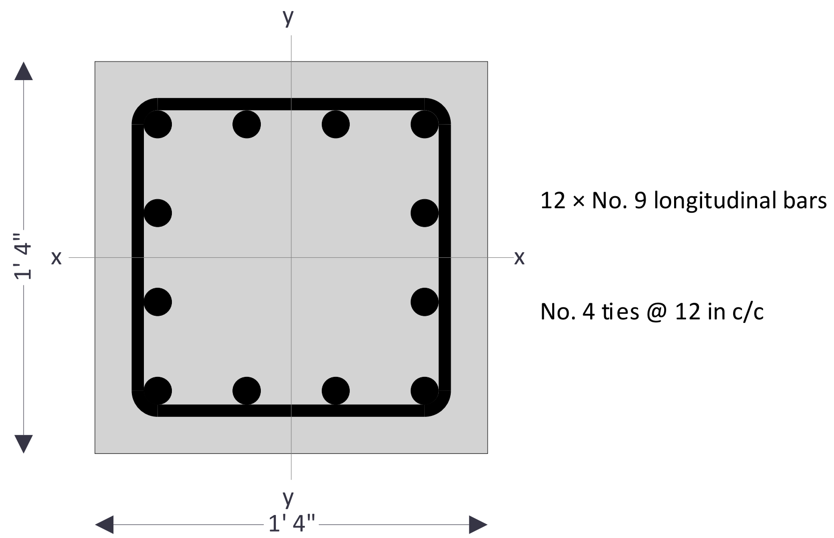
Area of shear reinforcement provided $A_{sv,prov}$ = **0.442** in^2/ft

TT T T T Tϕ T T

Tekla Tedds GAI Consultants, Inc. 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA 16066	Project I-10 Rest Area			Job Ref. B190988.00	
	Section Center Area Moment Frame Columns			Sheet no./rev. 1	
	Calc. by NJA	Date 7/7/2020	Chk'd by	Date	App'd by	Date

RC RECTANGULAR COLUMN DESIGN (ACI318-14)

Tedds calculation version 2.2.02



Applied loads

Ultimate axial force acting on column	P_{u_act} = **42.86** kips
Ultimate smaller end moment about x axis	M_{1x_act} = **140.6** kips_ft
Ultimate larger end moment about x axis	M_{2x_act} = **171** kips_ft
Column curvature about x axis	single curvature
Ratio of DL moment to total moment	β_d = **0.600**

Geometry of column

Depth of column (larger dimension of column)	h = **16.0** in
Width of column (smaller dimension of column)	b = **16.0** in
Clear cover to reinforcement (both sides)	c_c = **1.5** in
Unsupported height of column about x axis	l_{ux} = **17.5** ft
Effective height factor about x axis	k_x = **1.00**
Column state about the x axis	Braced
Unsupported height of column about y axis	l_{uy} = **17.5** ft
Effective height factor about y axis	k_y = **1.00**
Column state about the y axis	Braced

Check on overall column dimensions

X X X XXXX

Reinforcement of column

Numbers of bars of longitudinal steel	N = **12**
Longitudinal steel bar diameter number	D_{bar_num} = **9**
Diameter of longitudinal bar	D_{long} = **1.128** in
Stirrup bar diameter number	D_{stir_num} = **4**
Diameter of stirrup bar	D_{stir} = **0.500** in
Specified yield strength of reinforcement	f_y = **60000** psi
Specified compressive strength of concrete	f'_c = **4000** psi

Modulus of elasticity of bar reinforcement E_s = **29×10^6** psi

Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1psi)^{1/2}$ = **3604997** psi

Yield strain $\varepsilon_y = f_y / E_s$ = **0.00207**

Ultimate design strain ε_c = **0.003** in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column $A_g = h \times b$ = **256.000** in^2

Area of steel $A_{st} = N \times (\pi \times D_{long}^2) / 4$ = **11.992** in^2

Minimum area of steel required $A_{st\_min} = 0.01 \times A_g$ = **2.560** in^2

X X X X X X X

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel $A_{st\_max} = 0.08 \times A_g$ = **20.480** in^2

X X X XX X X

Slenderness check about x axis

Radius of gyration $r_x = 0.3 \times h$ = **4.8** in

Actual slenderness ratio $s_{rx\_act} = k_x \times l_{ux} / r_x$ = **43.75**

Permissible slenderness ratio $s_{rx\_perm} = \min(34 - 12 \times (M_{1x\_act} / M_{2x\_act}),40)$ = **24.13**

X X X X X X

Magnified moments about x axis

Moment of inertia of section $I_{gx} = (b \times h^3) / 12$ = **5461.333** in^4

Euler's buckling load $P_{cx} = (\pi^2 / (k_x \times l_{ux})^2) \times (0.4 \times E_c \times I_{gx} / (1+\beta_d))$ = **1101.55** kips

Correction factor for actual to equiv. mmt.diagram $C_{mx} = 0.6 + (0.4 \times M_{1x\_act} / M_{2x\_act})$ = **0.929**

Moment magnifier $\delta_{nsx} = \max(C_{mx} / (1 - (P_{u\_act} / (0.75 \times P_{cx}))),1.0)$ = **1**

Magnified moment about x axis $M_{cx} = \delta_{nsx} \times M_{2x\_act}$ = **171** kip_ft

Minimum factored moment about x axis $M_{2x\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times h)$ = **3.86** kip_ft

Minimum magnified moment about x axis $M_{cx\_min} = \delta_{nsx} \times M_{2x\_min}$ = **3.86** kip_ft

Slenderness check about y axis

Radius of gyration $r_y = 0.3 \times b$ = **4.8** in

Actual slenderness ratio $s_{ry\_act} = k_y \times l_{uy} / r_y$ = **43.75**

Permissible slenderness ratio $s_{ry\_perm} = \min(34 - 12 \times (M_{1y\_act} / M_{2y\_act}),40)$ = **34**

X X X X X X

Magnified moments about y axis

Moment of inertia of section $I_{gy} = (h \times b^3) / 12$ = **5461.333** in^4

Euler's buckling load $P_{cy} = (\pi^2 / (k_y \times l_{uy})^2) \times (0.4 \times E_c \times I_{gy} / (1+\beta_d))$ = **1101.55** kips

Correction factor for actual to equiv. mmt.diagram C_{my} = 1.0

Moment magnifier $\delta_{nsy} = \max(C_{my} / (1 - (P_{u\_act} / (0.75 \times P_{cy}))),1.0)$ = **1.055**

Minimum factored moment about y axis $M_{2y\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times b)$ = **3.86** kip_ft

Minimum magnified moment about y axis $M_{cy\_min} = \delta_{nsy} \times M_{2y\_min}$ = **4.07** kip_ft

Axial load capacity of axially loaded column

Strength reduction factor ϕ = 0.65

Area of steel on compression face $A'_s = A_{st} / 2$ = **5.996** in^2

Area of steel on tension face $A_s = A_{st} / 2 =$ **5.996** in^2

Net axial load capacity of column $P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}) =$ **1239.315** kips

Ultimate axial load capacity of column $P_u = \phi \times P_n =$ **805.555** kips

XX X X X X X

Net moments for biaxial column

Assuming strength reduction factor $\phi = 0.65$

Net moment about major (X) axis $M_{nx} = M_{cx} / \phi =$ **263.08** kips_ft

Net moment about minor (Y) axis $M_{ny} = M_{cy\_min} / \phi =$ **6.26** kips_ft

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio $r_{xb} =$ **0.414**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.564** in

Spacing between bars $s = ((h - (2 \times d')))/((N/2)-1) =$ **2.174** in

Depth of tension steel $d_t = h - d' =$ **13.436** in

Depth of NA from extreme compression face $c_x = r_{xb} \times d_t =$ **5.558** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_x = \min((\beta_1 \times c_x), h) =$ **4.724** in

Yield strain in steel $\varepsilon_{sx} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_x =$ **0.836**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{xcon} = 0.85 \times f'_c \times b \times a_x =$ **256.995** kips

Moment carried by concrete

Moment carried by concrete $M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) =$ **120.743** kip_ft

Details of steel layer 1

Depth of layer $x_{x1} =$ **2.564** in

Strain of layer $\varepsilon_{x1} = \varepsilon_c \times (1 - x_{x1} / c_x) =$ **0.00162**

Stress in layer $\sigma_{x1} = \min(f_y, E_s \times \varepsilon_{x1}) - 0.85 \times f'_c =$ **43464.35** psi

Force carried by layer $P_{x1} = N_x \times A_{bar} \times \sigma_{x1} =$ **173.741** kips

Moment carried by steel layer $M_{x1} = P_{x1} \times ((h / 2) - x_{x1}) =$ **78.704** kip_ft

Details of steel layer 2

Depth of layer $x_{x2} =$ **6.188** in

Strain of layer $\varepsilon_{x2} = \varepsilon_c \times (1 - x_{x2} / c_x) =$ **-0.00034**

Stress in layer $\sigma_{x2} = \max(-1 \times f_y, E_s \times \varepsilon_{x2}) =$ **-9864.04** psi

Force carried by layer $P_{x2} = 2 \times A_{bar} \times \sigma_{x2} =$ **-19.715** kips

Moment carried by steel layer $M_{x2} = P_{x2} \times ((h / 2) - x_{x2}) =$ **-2.977** kip_ft

Details of steel layer 3

Depth of layer $x_{x3} =$ **9.812** in

Strain of layer $\varepsilon_{x3} = \varepsilon_c \times (1 - x_{x3} / c_x) =$ **-0.00230**

Stress in layer $\sigma_{x3} = \max(-1 \times f_y, E_s \times \varepsilon_{x3}) =$ **-60000.00** psi

Force carried by layer $P_{x3} = 2 \times A_{bar} \times \sigma_{x3} =$ **-119.919** kips

Moment carried by steel layer $M_{x3} = P_{x3} \times ((h / 2) - x_{x3}) =$ **18.108** kip_ft

Details of steel layer 4

Depth of layer $x_{x4} =$ **13.436** in

Strain of layer $\varepsilon_{x4} = \varepsilon_c \times (1 - x_{x4} / c_x) =$ **-0.00425**

Stress in layer $\sigma_{x4} = \max(-1 \times f_y, E_s \times \varepsilon_{x4}) =$ **-60000.00** psi

Force carried by layer $P_{x4} = N_x \times A_{bar} \times \sigma_{x4} =$ **-239.839** kips

Moment carried by steel layer $M_{x4} = P_{x4} \times ((h / 2) - x_{x4}) =$ **108.647** kip_ft

Force carried by steel

Sum of forces by steel $P_{xs} =$ **-205.7** kips

Total force carried by column

Nominal axial load strength $P_{nx} =$ **51.263** kips

Strength reduction factor $\phi_x =$ **0.836**

Ultimate axial load carrying capacity of column $P_{ux} = \phi_x \times P_{nx} =$ **42.868** kips

Total moment carried by column

Total moment carried by column $M_{ox} =$ **323.225** kip_ft

Ultimate moment strength capacity of column $M_{ux} = \phi_x \times M_{ox} =$ **270.293**kip_ft

Equivalent required uniaxial moment about x axis

Equivalent required uniaxial nominal moment $M_{nxe} = M_{nx} + M_{ny} \times h / b \times ((1 - \beta) / \beta) =$ **269.336** kip_ft

Equivalent required uniaxial ultimate moment $M_{uxe} = M_{nxe} \times \phi_x =$ **225.229** kip_ft

Check load capacity about the x axis

Factored axial load $P_{u\_act} =$ **42.9** kips

Ultimate axial capacity $P_{ux} =$ **42.9** kips

XX X X X X X X

Equivalent required uniaxial factored moment $M_{uxe} =$ **225.2** kip_ft

Ultimate moment capacity about the x axis $M_{ux} =$ **270.3** kip_ft

XX X X X X X X XX

Uniaxially loaded column about minor axis

Details of column cross-section

c/d_t ratio $r_{yb} =$ **0.414**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.564** in

Spacing between bars $s = ((b – (2\times d')))/ ((N/2)-1) =$ **2.174** in

Depth of tension steel $b_t = b - d' =$ **13.436** in

Depth of NA from extreme compression face $c_y = r_{yb} \times b_t =$ **5.558** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_y = \min((\beta_1 \times c_y), b) =$ **4.724** in

Yield strain in steel $\varepsilon_{sy} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_y =$ **0.836**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{ycon} = 0.85 \times f'_c \times h \times a_y$ = **256.995** kips

Moment carried by concrete

Moment carried by concrete $M_{ycon} = P_{ycon} \times ((b/2) – (a_y/2))$ = **120.743** kip_ft

Details of steel layer 1

Depth of layer x_{y1} = **2.564** in

Strain of layer $\varepsilon_{y1} = \varepsilon_c \times (1 - x_{y1} / c_y)$ = **0.00162**

Stress in layer $\sigma_{y1} = \min(f_y, E_s \times \varepsilon_{y1}) - 0.85 \times f'_c$ = **43464.35** psi

Force carried by layer $P_{y1} = N_y \times A_{bar} \times \sigma_{y1}$ = **173.741** kips

Moment carried by steel layer $M_{y1} = P_{y1} \times ((b / 2) - x_{y1})$ = **78.704** kip_ft

Details of steel layer 2

Depth of layer x_{y2} = **6.188** in

Strain of layer $\varepsilon_{y2} = \varepsilon_c \times (1 - x_{y2} / c_y)$ = **-0.00034**

Stress in layer $\sigma_{y2} = \max(-1 \times f_y, E_s \times \varepsilon_{y2})$ = **-9864.04** psi

Force carried by layer $P_{y2} = 2 \times A_{bar} \times \sigma_{y2}$ = **-19.715** kips

Moment carried by steel layer $M_{y2} = P_{y2} \times ((b / 2) - x_{y2})$ = **-2.977** kip_ft

Details of steel layer 3

Depth of layer x_{y3} = **9.812** in

Strain of layer $\varepsilon_{y3} = \varepsilon_c \times (1 - x_{y3} / c_y)$ = **-0.00230**

Stress in layer $\sigma_{y3} = \max(-1 \times f_y, E_s \times \varepsilon_{y3})$ = **-60000.00** psi

Force carried by layer $P_{y3} = 2 \times A_{bar} \times \sigma_{y3}$ = **-119.919** kips

Moment carried by steel layer $M_{y3} = P_{y3} \times ((b / 2) - x_{y3})$ = **18.108** kip_ft

Details of steel layer 4

Depth of layer x_{y4} = **13.436** in

Strain of layer $\varepsilon_{y4} = \varepsilon_c \times (1 - x_{y4} / c_y)$ = **-0.00425**

Stress in layer $\sigma_{y4} = \max(-1 \times f_y, E_s \times \varepsilon_{y4})$ = **-60000.00** psi

Force carried by layer $P_{y4} = N_y \times A_{bar} \times \sigma_{y4}$ = **-239.839** kips

Moment carried by steel layer $M_{y4} = P_{y4} \times ((b / 2) - x_{y4})$ = **108.647** kip_ft

Force carried by steel

Sum of forces by steel P_{ys} = **-205.7** kips

Total force carried by column

Nominal axial load strength P_{ny} = **51.263** kips

Strength reduction factor ϕ_y = **0.836**

Ultimate axial load carrying capacity of column $P_{uy} = \phi_y \times P_{ny}$ = **42.868** kips

Moment carried by biaxial column minor axis

Nominal moment strength M_{oy} = **323.225** kip_ft

Contour beta factor

Contour beta factor β = **0.500**

M_{nx}_upon_$M_{ox1} = M_{nx} / M_{ox}$ = **0.814**

	M_{ny}_upon_M_{oy} = **0.186**
Net moment along minor axis resisted by column	M_{ny1} = M_{oy} × (M_{ny}_upon_M_{oy}) = **60.120** kip_ft
Ultimate moment along minor axis	M_{uy} = M_{ny1} × ϕ_y = **50.275** kip_ft

Check load capacity about the y axis

Factored axial load	P_{u_act} = **42.9** kips
Ultimate axial capacity	P_{uy} = **42.9** kips
	XX X X X X X X
Factored moment about the y axis	M_{uy_max} = ϕ × M_{ny} = **4.1** kip_ft
Ultimate moment capacity about the y axis	M_{uy} = **50.3** kip_ft
	XX X X X X X X XX

Design of column ties - 25.7.2

Spacing of lateral ties	s_{v_ties} = **12.000** in
16 times longitudinal bar diameter	s_{v1} = 16 × D_{long} = **18.048** in
48 times tie bar diameter	s_{v2} = 48 × D_{stir} = **24.000** in
Least column dimension	s_{v3} = min (h,b) = **16.000** in
Required tie spacing	s = min(s_{v1},s_{v2},s_{v3}) = **16.000** in

X XXX X

Restroom area Foundation

References: 1. ACI 318-14

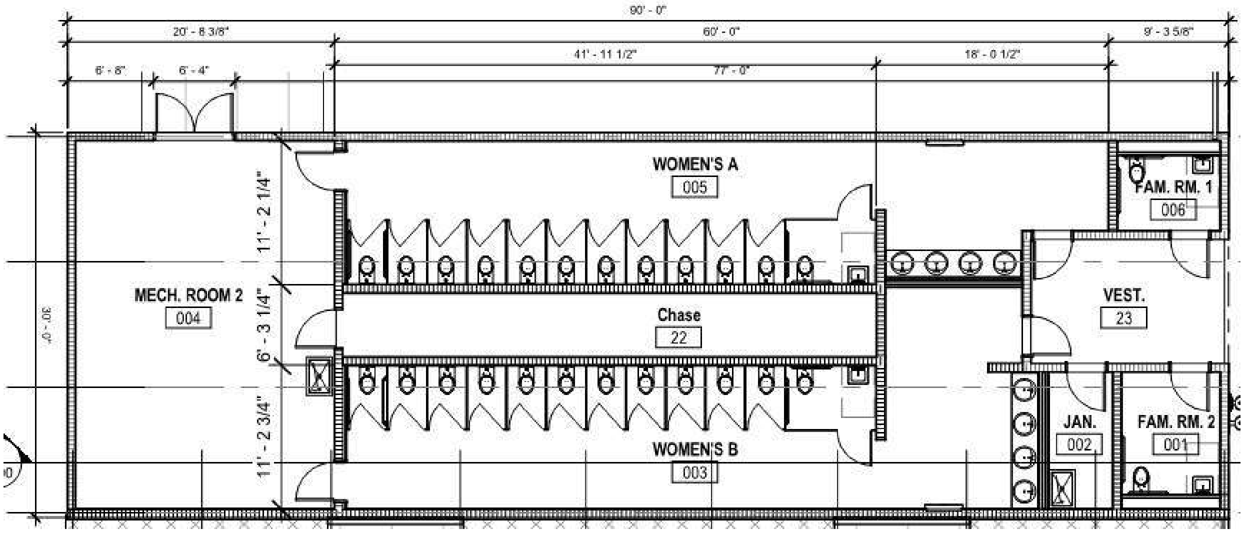


Figure 1

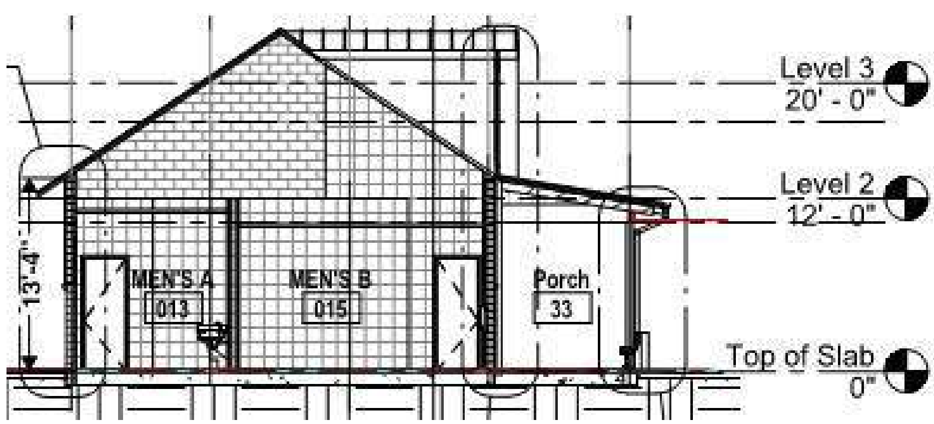


Figure 2

Building Parameters

$L_{bldg} := 90ft$ $B_{bldg} := 30ft$ $H_{eave} := 13ft + 4in$ $H_{mean} := 18ft + 4in$

$t_{wall} := 8in$ $t_{slab} := 4in$

Building makeup is assumed to be 1 1/2" metal deck supporting 2" rigid insulation and a standing seam cover.
The roof is supported by light gauge trusses on 2' centers that span to CMU bearing walls

Walls are supported on continuous spread

There is a 2'-0" overhang on one side and a 10'-0" porch roof on the other side

Load Generation

$DL_{roof} := 20psf$ This will include the roof and supporting trusses

$LL_{roof} := 20psf$ Minimum

$Roof_{ovhg1} := 2ft$ $Roof_{ovhg2} := 5ft$ $Wt_{CMU} := 61psf$ Assiming reinforcement at 24" on center

Side Wall Loads

$$W_{DLroof1} := \left(\frac{B_{bldg}}{2} + Roof_{ovhg1}\right) \cdot DL_{roof} = 340 \cdot \frac{lb}{ft} \qquad W_{DLroof2} := \left(\frac{B_{bldg}}{2} + Roof_{ovhg2}\right) \cdot DL_{roof} = 400 \cdot \frac{lb}{ft}$$

$$W_{LLroof1} := \left(\frac{B_{bldg}}{2} + Roof_{ovhg1}\right) \cdot LL_{roof} = 340 \cdot \frac{lb}{ft} \qquad W_{LLroof2} := \left(\frac{B_{bldg}}{2} + Roof_{ovhg2}\right) \cdot LL_{roof} = 400 \cdot \frac{lb}{ft}$$

$$Wt_{brgwall} := H_{eave} \cdot Wt_{CMU} = 813.333 \cdot \frac{lb}{ft}$$

End Wall Loads

CMU wall extends to roof line $\gamma_{conc} := 150pcf$

Assume a 1'-0" concrete raked bond beam along the top

$$A_{\Delta} := \frac{B_{bldg} \cdot 10ft}{2} = 150\ ft^2$$ Triangular wall area above eave

$$A_{CMU} := \frac{\left(B_{bldg} - 2ft\right) \cdot 9ft}{2} = 126\ ft^2$$ Triangular CMU wall area above eave

$$Wt_{Addcmu} := A_{CMU} \cdot Wt_{CMU} = 7686\ lbf$$

$$Wt_{Addconc} := \left(A_{\Delta} - A_{CMU}\right) \cdot \gamma_{conc} \cdot t_{wall} = 2400 \cdot lb$$

$$Wt_{Add} := Wt_{Addcmu} + Wt_{Addconc} = 10086 \cdot lb$$

$$Wt_{endwall} := \frac{Wt_{Add}}{B_{bldg}} = 336.2 \cdot \frac{lb}{ft}$$

Footing Loads

Side Wall

$$W_{asd1} := W_{DLroof1} + Wt_{brgwall} + W_{LLroof1} = 1493.333 \cdot \frac{lb}{ft}$$

$$W_{asd2} := W_{DLroof2} + Wt_{brgwall} + W_{LLroof2} = 1613.333 \cdot \frac{lb}{ft}$$

End Wall

$$W_{asdend} := Wt_{endwall} + Wt_{brgwall} = 1149.533 \cdot \frac{lb}{ft}$$

Additional DL below grade

$$H_{below} := 1ft + 4in \qquad \gamma_{soil} := 110pcf$$

$$CMU := H_{below} \cdot Wt_{CMU} = 81.333 \cdot \frac{lb}{ft}$$

Assume 2'-4" wide footer

$$B_{ftr} := 2ft + 4in$$

$$SOIL := \left(B_{ftr} - t_{wall}\right) \cdot \left(H_{below} - t_{slab}\right) \cdot \gamma_{soil} = 183.333 \cdot \frac{lb}{ft}$$

$$CONC := \frac{\left(B_{ftr} - t_{wall}\right)}{2} \cdot t_{slab} \cdot \gamma_{conc} = 42 \cdot \frac{lb}{ft}$$

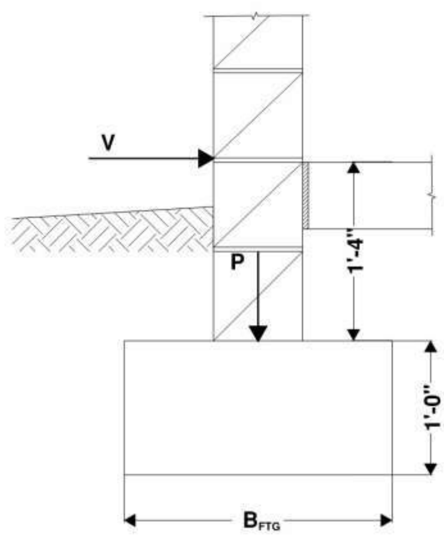


Figure 3

For Wind Load:

Assume wall is pinned at the top and bottom

Roof diaphragm takes the top reaction to shear walls

$WL_{wall} := 20.869psf$ Max from the wind load analysis

$R_{flr} := \frac{WL_{wall} \cdot H_{eave}}{2} = 139.127 \cdot \frac{lb}{ft}$ Shear per foot of wall

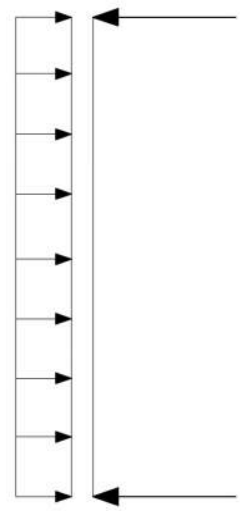


Figure 4

Load Combinations (Side Wall)

LC3 DL+LL$_r$

LC 5 DL+0.6WL

LC6a DL+0.75LL$_r$+0.45WL

LC7 0.6DL+0.6WL

$P_{DL1} := W_{DLroof1} + Wt_{brgwall} + CMU + SOIL + CONC = 1459.667 \cdot \frac{lb}{ft}$

$P_{LL1} := W_{LLroof1} = 340 \cdot \frac{lb}{ft}$

$P_{DL2} := W_{DLroof2} + Wt_{brgwall} + CMU + SOIL + CONC = 1519.667 \cdot \frac{lb}{ft}$

$P_{LL2} := W_{LLroof2} = 400 \cdot \frac{lb}{ft}$

$P_{WLn1} := -688 \frac{lb}{ft}$ $P_{WLp1} := 158 \frac{lb}{ft}$

$P_{WLn2} := P_{WLn1} - 36.699psf \cdot 5ft = -871.495 \cdot \frac{lb}{ft}$ $P_{WLp2} := P_{WLp1} + 39.6psf \cdot 5ft = 356 \cdot \frac{lb}{ft}$

Load Combinations

$P_{LC31} := P_{DL1} + P_{LL1} = 1800 \cdot \frac{lb}{ft}$

$P_{LC32} := P_{DL2} + P_{LL2} = 1920 \cdot \frac{lb}{ft}$

$P_{LC5n1} := P_{DL1} + 0.6 \cdot P_{WLn1} = 1046.867 \cdot \frac{lb}{ft}$

$P_{LC5p1} := P_{DL1} + 0.6P_{WLp1} = 1554.467 \cdot \frac{lb}{ft}$

$$P_{LC5n2} := P_{DL2} + 0.6 \cdot P_{WLn2} = 996.770 \cdot \frac{lb}{ft}$$

$$P_{LC5p2} := P_{DL2} + 0.6P_{WLp2} = 1733.267 \cdot \frac{lb}{ft}$$

$$P_{LC6an1} := P_{DL1} + 0.75 \cdot P_{LL1} + 0.45P_{WLn1} = 1405.067 \cdot \frac{lb}{ft}$$

$$P_{LC6ap1} := P_{DL1} + 0.75 \cdot P_{LL1} + 0.45P_{WLp1} = 1785.767 \cdot \frac{lb}{ft}$$

$$P_{LC6an2} := P_{DL2} + 0.75 \cdot P_{LL2} + 0.45P_{WLn2} = 1427.494 \cdot \frac{lb}{ft}$$

$$P_{LC6ap2} := P_{DL2} + 0.75 \cdot P_{LL2} + 0.45P_{WLp2} = 1979.867 \cdot \frac{lb}{ft}$$

$$P_{LC7n1} := 0.6 \cdot P_{DL1} + 0.6 \cdot P_{WLn1} = 463 \cdot \frac{lb}{ft}$$

$$P_{LC7p1} := 0.6 \cdot P_{DL1} + 0.6 \cdot P_{WLp1} = 970.6 \cdot \frac{lb}{ft}$$

$$P_{LC7n2} := 0.6 \cdot P_{DL2} + 0.6 \cdot P_{WLn2} = 388.903 \cdot \frac{lb}{ft}$$

$$P_{LC7p2} := 0.6 \cdot P_{DL2} + 0.6 \cdot P_{WLp2} = 1125.4 \cdot \frac{lb}{ft}$$

By inspection LC6ap will govern for maximum pressure and LC7n will govern for minimum pressure

PROPOSED RESTROOM FACILITIES BEARING CAPACITY CALCULATIONS
SOIL BORINGS SS-4 AND SS-5

MODULUS OF SUBGRADE REACTION (Scott 1981)

$K_s := 6 \cdot N_{cor} \cdot \frac{ton}{ft^3}$ $\quad \boxed{K_s = 84000 \frac{lb}{ft^3}}$ *<~ Modulus of Subgrade Reaction*

ALLOWABLE BEARING RESISTANCE (Munfakh 2001)

$q_{n\_munfakh} := c \cdot N_c \cdot s_c + \gamma \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma \cdot C_{w\gamma}$

$q_{n\_munfakh} = 5007.2\ psf$

$\phi_b := 0.45$ *<~ Resistance Factor (LRFD)*

$q_{a\_munfakh} := \phi_b \cdot q_{n\_munfakh}$

$q_{a\_munfakh} = 2253\ psf$ *<~ Allowable Bearing Resistance*

$B_{ftg} := 1ft + 4in = 1.333\,ft$

$A_{ftg} := 1ft \cdot B_{ftg} = 1.333 \cdot ft^2$ $\qquad S_{ftg} := \frac{1ft \cdot (B_{ftg})^2}{6} = 0.296 \cdot ft^3$

Soil Pressure

$f_{pLC6apmax1} := \frac{1ft \cdot P_{LC6ap1}}{A_{ftg}} = 1339.325 \cdot psf$ $\qquad < f_{pmax} = 2253\ psf$ **OK**

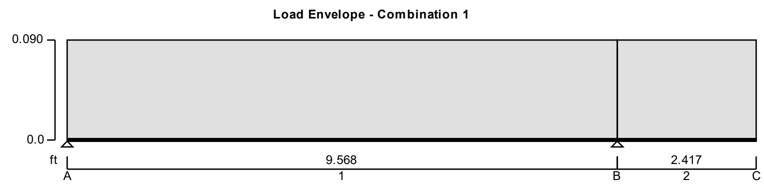
Pouch Area Beam and Columns

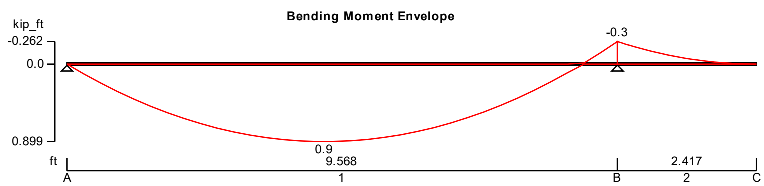
Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas				Job Ref. B190988.00	
	Section Porch Roof Rafters LC 6a, DL+.75LL+.45WL				Sheet no./rev. 1	
	Calc. by EJM	Date 5/19/2020	Chk'd by	Date	App'd by	Date

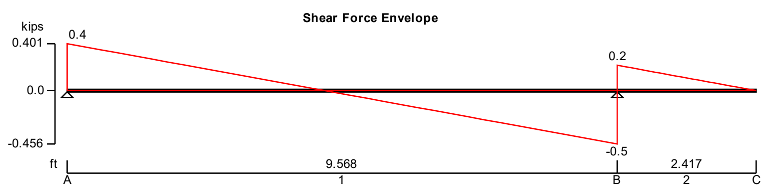
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04







Applied loading

Beam loads

Dead	Dead full UDL 24 lb/ft
Live	Live full UDL 40 lb/ft
Wind	Wind full UDL 79 lb/ft

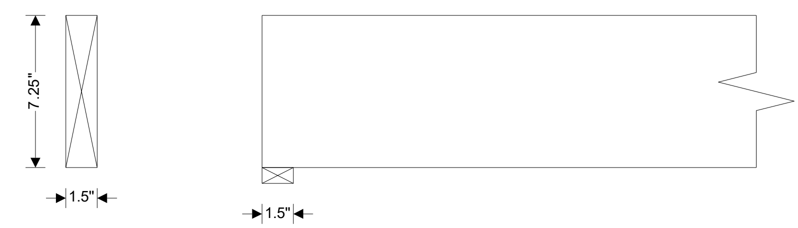
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Span 1	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Wind × 0.45

Span 2	Dead × 1.00	
	Live × 0.75	
	Wind × 0.45	
Support C	Dead × 1.00	
	Live × 0.75	
	Wind × 0.45	

Analysis results

Maximum moment	M_{max} = **899** lb_ft	M_{min} = **-262** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **899** lb_ft	
Maximum shear	F_{max} = **401** lb	F_{min} = **-456** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **456** lb	
Total load on member	W_{tot} = **1074** lb	
Reaction at support A	R_{A_max} = **401** lb	R_{A_min} = **401** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **107** lb	
Unfactored live load reaction at support A	R_{A_Live} = **179** lb	
Unfactored wind load reaction at support A	R_{A_Wind} = **355** lb	
Reaction at support B	R_{B_max} = **673** lb	R_{B_min} = **673** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **180** lb	
Unfactored live load reaction at support B	R_{B_Live} = **300** lb	
Unfactored wind load reaction at support B	R_{B_Wind} = **594** lb	
Reaction at support C	R_{C_max} = **0** lb	R_{C_min} = **0** lb
Unfactored dead load reaction at support C	R_{C_Dead} = **0** lb	
Unfactored live load reaction at support C	R_{C_Live} = **0** lb	
Unfactored wind load reaction at support C	R_{C_Wind} = **0** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **8** in
Dressed depth of sections	d = **7.25** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 8" wide
Bending parallel to grain	F_b = **925** lb/in^2
Tension parallel to grain	F_t = **550** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2

Compression perpendicular to grain F_{c_perp} = **565** lb/in^2

Shear parallel to grain F_v = **175** lb/in^2

Modulus of elasticity E = **1400000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **510000** lb/in^2

Mean shear modulus $G_{def} = E / 16$ = **87500** lb/in^2

Member details

Service condition **Wet**

Length of bearing L_b = **1.5** in

Load duration **Permanent**

Section properties

Cross sectional area of member $A = N \times b \times d$ = **10.87** in^2

Section modulus $S_x = N \times b \times d^2 / 6$ = **13.14** in^3

$S_y = d \times (N \times b)^2 / 6$ = **2.72** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **47.63** in^4

$I_y = d \times (N \times b)^3 / 12$ = **2.04** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **0.90**

Temperature factor - Table 2.3.3 C_t = **1.00**

Size factor for bending - Table 4B C_{Fb} = **1.00**

Size factor for tension - Table 4B C_{Ft} = **1.00**

Size factor for compression - Table 4B C_{Fc} = **1.00**

Flat use factor - Table 4B C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

Incising factor for bending, shear, tension & compression - Table 4.3.8

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 $C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.25**

Wet service factor for bending - Table 4B C_{Mb} = **1.00**

Wet service factor for tension - Table 4B C_{Mt} = **1.00**

Wet service factor for compression - Table 4B C_{Mc} = **0.80**

Wet service factor for perpendicular compression - Table 4B

C_{Mc_perp} = **0.67**

Wet service factor for shear - Table 4B C_{Mv} = **0.97**

Wet service factor for modulus of elasticity - Table 4B

C_{ME} = **0.90**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **4.00**

- Beam is fully restrained

Beam stability factor - cl.3.3.3 C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_{Mc\_perp} \times C_t \times C_i \times C_b$ = **473** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b)$ = **299** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.632**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **833** lb/in^2

Actual bending stress $f_b = M / S_x =$ **821** lb/in^2

$f_b / F_b' =$ **0.986**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i =$ **153** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **63** lb/in^2

$f_v / F_v' =$ **0.412**

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1260000** lb/in^2

Design deflection $\delta_{adm} =$ **0.479** in

Bending deflection $\delta_{b\_s2} =$ **0.253** in

Shear deflection $\delta_{v\_s2} =$ **0.000** in

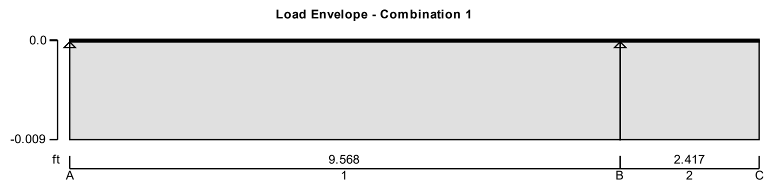
Total deflection $\delta_a = \delta_{b\_s2} + \delta_{v\_s2} =$ **0.253** in

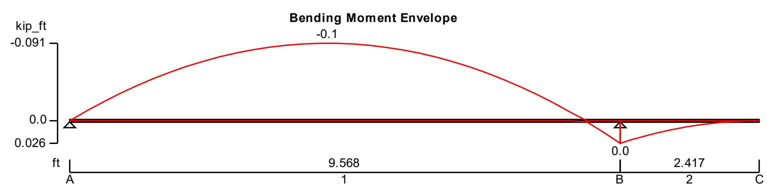
$\delta_a / \delta_{adm} =$ **0.529**

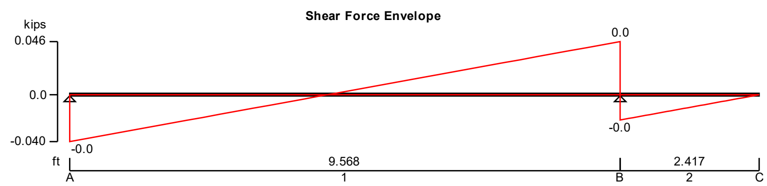
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04







Applied loading

Beam loads

Dead	Dead full UDL 24 lb/ft
Wind	Wind full UDL -73 lb/ft

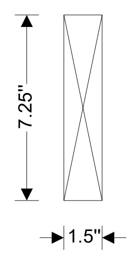
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Span 1	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Span 2	Dead × 1.00

		Live × 0.75
		Wind × 0.45
	Support C	Dead × 1.00
		Live × 0.75
		Wind × 0.45

Analysis results

Maximum moment	M_{max} = **26** lb_ft	M_{min} = **-91** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **91** lb_ft	
Maximum shear	F_{max} = **46** lb	F_{min} = **-40** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **46** lb	
Total load on member	W_{tot} = **108** lb	
Reaction at support A	R_{A_max} = **-40** lb	R_{A_min} = **-40** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **107** lb	
Unfactored wind load reaction at support A	R_{A_Wind} = **-329** lb	
Reaction at support B	R_{B_max} = **-68** lb	R_{B_min} = **-68** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **180** lb	
Unfactored wind load reaction at support B	R_{B_Wind} = **-551** lb	
Reaction at support C	R_{C_max} = **0** lb	R_{C_min} = **0** lb
Unfactored dead load reaction at support C	R_{C_Dead} = **0** lb	
Unfactored wind load reaction at support C	R_{C_Wind} = **0** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **8** in
Dressed depth of sections	d = **7.25** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 8" wide
Bending parallel to grain	F_b = **925** lb/in^2
Tension parallel to grain	F_t = **550** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **565** lb/in^2
Shear parallel to grain	F_v = **175** lb/in^2
Modulus of elasticity	E = **1400000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **510000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **87500** lb/in^2

Member details

Service condition **Wet**

Length of bearing L_b = **1.5** in

Load duration **Permanent**

Section properties

Cross sectional area of member $A = N \times b \times d$ = **10.87** in^2

Section modulus $S_x = N \times b \times d^2 / 6$ = **13.14** in^3

$S_y = d \times (N \times b)^2 / 6$ = **2.72** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **47.63** in^4

$I_y = d \times (N \times b)^3 / 12$ = **2.04** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **0.90**

Temperature factor - Table 2.3.3 C_t = **1.00**

Size factor for bending - Table 4B C_{Fb} = **1.00**

Size factor for tension - Table 4B C_{Ft} = **1.00**

Size factor for compression - Table 4B C_{Fc} = **1.00**

Flat use factor - Table 4B C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

Incising factor for bending, shear, tension & compression - Table 4.3.8

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 $C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.25**

Wet service factor for bending - Table 4B C_{Mb} = **1.00**

Wet service factor for tension - Table 4B C_{Mt} = **1.00**

Wet service factor for compression - Table 4B C_{Mc} = **0.80**

Wet service factor for perpendicular compression - Table 4B

C_{Mc_perp} = **0.67**

Wet service factor for shear - Table 4B C_{Mv} = **0.97**

Wet service factor for modulus of elasticity - Table 4B

C_{ME} = **0.90**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **4.00**

- Beam is fully restrained

Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **833** lb/in^2

Actual bending stress $f_b = M / S_x$ = **83** lb/in^2

f_b / F_b' = **0.099**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i$ = **153** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **6** lb/in^2

$f_v / F_v' =$ **0.041**

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1260000** lb/in^2

Design deflection $\delta_{adm} =$ **0.479** in

Bending deflection $\delta_{b\_s2} =$ **0.087** in

Shear deflection $\delta_{v\_s2} =$ **0.000** in

Total deflection $\delta_a = \delta_{b\_s2} + \delta_{v\_s2} =$ **0.087** in

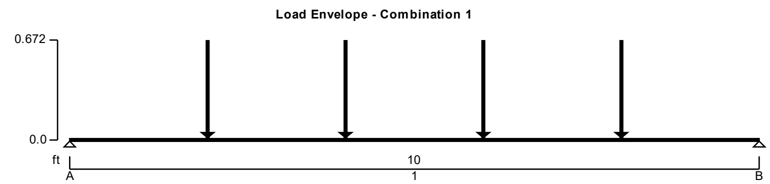
$\delta_a / \delta_{adm} =$ **0.182**

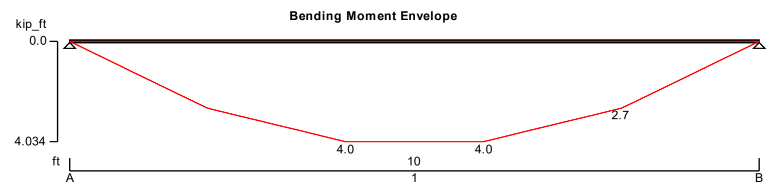
Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas			Job Ref. B190988.00	
	Section Porch Rafters Support Beam, LC6a, DL+0.75LL+0.45WL			Sheet no./rev. 1	
	Calc. by EJM	Date 5/19/2020	Chk'd by	Date	App'd by	Date

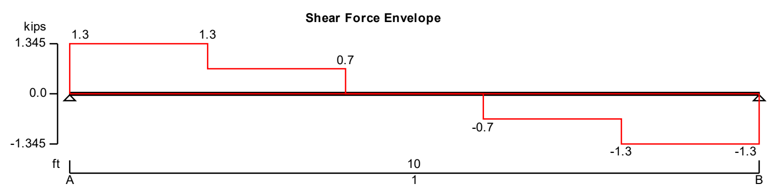
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04







Applied loading

Beam loads

Dead	Dead point load 180 lb at 24.00 in
Dead	Dead point load 180 lb at 48.00 in
Dead	Dead point load 180 lb at 72.00 in
Dead	Dead point load 180 lb at 96.00 in
Wind	Wind point load 594 lb at 24.00 in
Wind	Wind point load 594 lb at 48.00 in
Wind	Wind point load 594 lb at 72.00 in
Wind	Wind point load 594 lb at 96.00 in
Live	Live point load 300 lb at 24.00 in
Live	Live point load 300 lb at 48.00 in
Live	Live point load 300 lb at 72.00 in
Live	Live point load 300 lb at 96.00 in

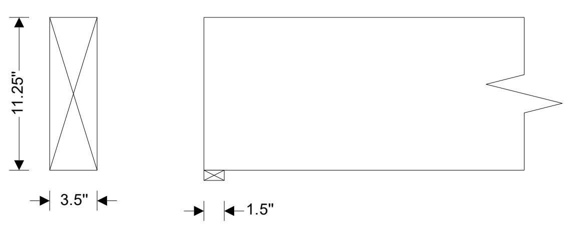
Load combinations

Load combination 1	Support A	Dead × 1.00

		Live × 0.75
		Wind × 0.45
	Span 1	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Wind × 0.45

Analysis results

Maximum moment	M_{max} = **4034** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4034** lb_ft	
Maximum shear	F_{max} = **1345** lb	F_{min} = **-1345** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1345** lb	
Total load on member	W_{tot} = **2689** lb	
Reaction at support A	R_{A_max} = **1345** lb	R_{A_min} = **1345** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **360** lb	
Unfactored live load reaction at support A	R_{A_Live} = **600** lb	
Unfactored wind load reaction at support A	R_{A_Wind} = **1188** lb	
Reaction at support B	R_{B_max} = **1345** lb	R_{B_min} = **1345** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **360** lb	
Unfactored live load reaction at support B	R_{B_Live} = **600** lb	
Unfactored wind load reaction at support B	R_{B_Wind} = **1188** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **4** in
Dressed breadth of sections	b = **3.5** in
Nominal depth of sections	d_{nom} = **12** in
Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **3.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 12" wide
Bending parallel to grain	F_b = **750** lb/in^2
Tension parallel to grain	F_t = **450** lb/in^2
Compression parallel to grain	F_c = **1250** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **565** lb/in^2
Shear parallel to grain	F_v = **175** lb/in^2

Modulus of elasticity $E = \mathbf{1400000}$ lb/in^2

Modulus of elasticity, stability calculations $E_{min} = \mathbf{510000}$ lb/in^2

Mean shear modulus $G_{def} = E / 16 = \mathbf{87500}$ lb/in^2

Member details

Service condition **Wet**

Length of bearing $L_b = \mathbf{1.5}$ in

Load duration **Permanent**

Section properties

Cross sectional area of member $A = N \times b \times d = \mathbf{39.37}$ in^2

Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{73.83}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{22.97}$ in^3

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{415.28}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{40.20}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{0.90}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4B $C_{Fb} = \mathbf{1.10}$

Size factor for tension - Table 4B $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4B $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4B $C_{fu} = \mathbf{1.10}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Wet service factor for bending - Table 4B $C_{Mb} = \mathbf{1.00}$

Wet service factor for tension - Table 4B $C_{Mt} = \mathbf{1.00}$

Wet service factor for compression - Table 4B $C_{Mc} = \mathbf{0.80}$

Wet service factor for perpendicular compression - Table 4B

$C_{Mc\_perp} = \mathbf{0.67}$

Wet service factor for shear - Table 4B $C_{Mv} = \mathbf{0.97}$

Wet service factor for modulus of elasticity - Table 4B

$C_{ME} = \mathbf{0.90}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{3.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_{Mc\_perp} \times C_t \times C_i \times C_b = \mathbf{379}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) = \mathbf{256}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.677}$

Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas			Job Ref. B190988.00	
	Section Porch Rafters Support Beam, LC6a, DL+0.75LL+0.45WL			Sheet no./rev. 4	
	Calc. by EJM	Date 5/19/2020	Chk'd by	Date	App'd by	Date

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **743** lb/in^2

Actual bending stress $f_b = M / S_x =$ **656** lb/in^2

$f_b / F_b' =$ **0.883**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i =$ **153** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **51** lb/in^2

$f_v / F_v' =$ **0.335**

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1260000** lb/in^2

Design deflection $\delta_{adm} =$ **0.333** in

Bending deflection $\delta_{b\_s1} =$ **0.223** in

Shear deflection $\delta_{v\_s1} =$ **0.027** in

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.250** in

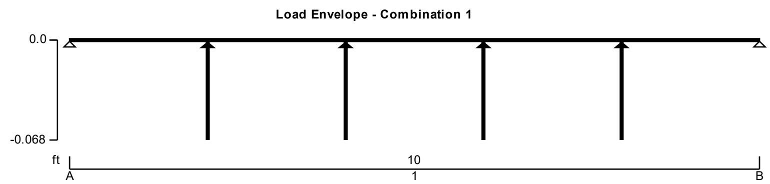
$\delta_a / \delta_{adm} =$ **0.752**

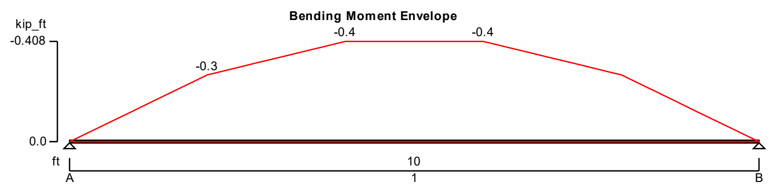
GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas			Job Ref. B190988.00	
	Section Porch Rafters Support Beam, LC7, 0.6DL+0.6WL			Sheet no./rev. 1	
	Calc. by EJM	Date 5/19/2020	Chk'd by	Date	App'd by	Date

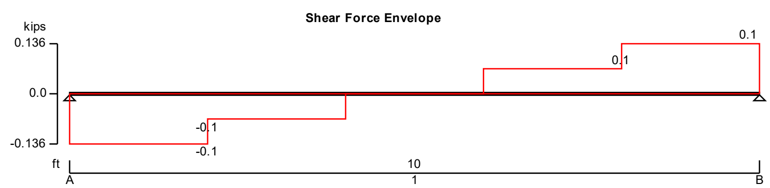
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04







Applied loading

Beam loads

Dead	Dead point load 180 lb at 24.00 in
Dead	Dead point load 180 lb at 48.00 in
Dead	Dead point load 180 lb at 72.00 in
Dead	Dead point load 180 lb at 96.00 in
Wind	Wind point load -551 lb at 24.00 in
Wind	Wind point load -551 lb at 48.00 in
Wind	Wind point load -551 lb at 72.00 in
Wind	Wind point load -551 lb at 96.00 in

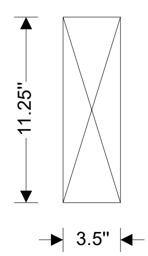
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 0.75
		Wind × 0.45
	Span 1	Dead × 1.00
		Live × 0.75

		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Wind × 0.45

Analysis results

Maximum moment	M_{max} = **0** lb_ft	M_{min} = **-408** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **408** lb_ft	
Maximum shear	F_{max} = **136** lb	F_{min} = **-136** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **136** lb	
Total load on member	W_{tot} = **272** lb	
Reaction at support A	R_{A_max} = **-136** lb	R_{A_min} = **-136** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **360** lb	
Unfactored wind load reaction at support A	R_{A_Wind} = **-1102** lb	
Reaction at support B	R_{B_max} = **-136** lb	R_{B_min} = **-136** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **360** lb	
Unfactored wind load reaction at support B	R_{B_Wind} = **-1102** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **4** in
Dressed breadth of sections	b = **3.5** in
Nominal depth of sections	d_{nom} = **12** in
Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **3.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 12" wide
Bending parallel to grain	F_b = **750** lb/in^2
Tension parallel to grain	F_t = **450** lb/in^2
Compression parallel to grain	F_c = **1250** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **565** lb/in^2
Shear parallel to grain	F_v = **175** lb/in^2
Modulus of elasticity	E = **1400000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **510000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **87500** lb/in^2

Member details

Service condition	**Wet**
Length of bearing	L_b = **1.5** in

Load duration **Permanent**

Section properties

Cross sectional area of member $A = N \times b \times d = \mathbf{39.37}$ in^2

Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{73.83}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{22.97}$ in^3

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{415.28}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{40.20}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{0.90}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4B $C_{Fb} = \mathbf{1.10}$

Size factor for tension - Table 4B $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4B $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4B $C_{fu} = \mathbf{1.10}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Wet service factor for bending - Table 4B $C_{Mb} = \mathbf{1.00}$

Wet service factor for tension - Table 4B $C_{Mt} = \mathbf{1.00}$

Wet service factor for compression - Table 4B $C_{Mc} = \mathbf{0.80}$

Wet service factor for perpendicular compression - Table 4B

$C_{Mc\_perp} = \mathbf{0.67}$

Wet service factor for shear - Table 4B $C_{Mv} = \mathbf{0.97}$

Wet service factor for modulus of elasticity - Table 4B

$C_{ME} = \mathbf{0.90}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{3.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{743}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{66}$ lb/in^2

$f_b / F_b' = \mathbf{0.089}$

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i = \mathbf{153}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{5}$ lb/in^2

$f_v / F_v' = \mathbf{0.034}$

Deflection - cl.3.5.1

Modulus of elasticity for deflection — $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1260000** lb/in^2

Design deflection — δ_{adm} = **0.333** in

Bending deflection — δ_{b_s1} = **0.077** in

Shear deflection — δ_{v_s1} = **0.020** in

Total deflection — $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.098** in

δ_a / δ_{adm} = **0.293**

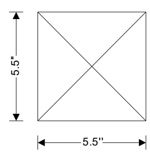
STRUCTURAL WOOD BEAM DESIGN (NDS)

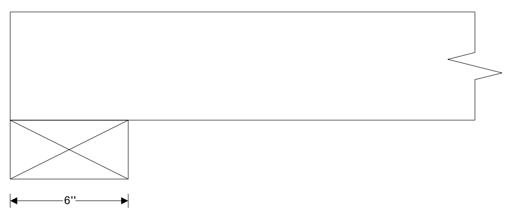
In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04

Analysis results

Design moment in major axis	M_x = **1080** lb_ft
Design shear	F = **135** lb
Maximum reaction	R = **3363** lb





Sawn lumber section details

Nominal breadth of sections	b_{nom} = **6** in
Dressed breadth of sections	b = **5.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **5.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 5" x 5" and larger
Bending parallel to grain	F_b = **850** lb/in^2
Tension parallel to grain	F_t = **550** lb/in^2
Compression parallel to grain	F_c = **525** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **375** lb/in^2
Shear parallel to grain	F_v = **165** lb/in^2
Modulus of elasticity	E = **1080000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **440000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **67500** lb/in^2

Member details

Service condition	**Wet**
Length of bearing	L_b = **6** in
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **8** ft
Effective length factor in x-axis	K_x = **2.1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **16.8** ft
Unbraced length in y-axis	L_y = **10.333** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **10.333** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **30.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **27.73** in^3

$S_y = d \times (N \times b)^2 / 6 =$ **27.73** in^3

Second moment of area $I_x = N \times b \times d^3 / 12 =$ **76.26** in^4

$I_y = d \times (N \times b)^3 / 12 =$ **76.26** in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D =$ **1.00**

Temperature factor - Table 2.3.3 $C_t =$ **1.00**

Size factor for bending - Table 4D $C_{Fb} =$ **1.00**

Size factor for tension - Table 4D $C_{Ft} =$ **1.00**

Size factor for compression - Table 4D $C_{Fc} =$ **1.00**

Flat use factor - Table 4D $C_{fu} =$ **1.00**

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} =$ **1.00**

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i =$ **1.00**

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} =$ **1.00**

Repetitive member factor - cl.4.3.9 $C_r =$ **1.00**

Bearing area factor - cl.3.10.4 $C_b =$ **1.00**

Wet service factor for bending - Table 4D $C_{Mb} =$ **1.00**

Wet service factor for tension - Table 4D $C_{Mt} =$ **1.00**

Wet service factor for compression - Table 4D $C_{Mc} =$ **0.91**

Wet service factor for perpendicular compression - Table 4D

$C_{Mc\_perp} =$ **0.67**

Wet service factor for shear - Table 4D $C_{Mv} =$ **1.00**

Wet service factor for modulus of elasticity - Table 4D

$C_{ME} =$ **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} =$ **440000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i =$ **478** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2 =$ **269** lb/in^2

$c =$ **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} =$ **0.48**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) =$ **1.00**

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L =$ **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_{Mc\_perp} \times C_t \times C_i \times C_b =$ **251** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R / (N \times b \times L_b) =$ **102** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.406**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **850** lb/in^2

Actual bending stress $f_b = M_x / S_x =$ **467** lb/in^2

$f_b / F_b' =$ **0.550**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i =$ **165** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **7** lb/in^2

$f_v / F_v' =$ **0.041**

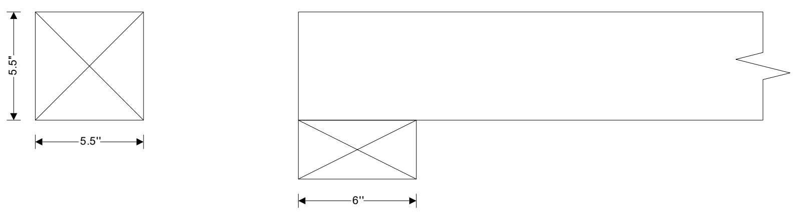
STRUCTURAL WOOD BEAM DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.04

Analysis results

Design moment in major axis	M_x = **1080** lb_ft
Design shear	F = **135** lb
Maximum reaction	R = **340** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **6** in
Dressed breadth of sections	b = **5.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **5.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 5" x 5" and larger
Bending parallel to grain	F_b = **850** lb/in^2
Tension parallel to grain	F_t = **550** lb/in^2
Compression parallel to grain	F_c = **525** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **375** lb/in^2
Shear parallel to grain	F_v = **165** lb/in^2
Modulus of elasticity	E = **1080000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **440000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **67500** lb/in^2

Member details

Service condition	**Wet**
Length of bearing	L_b = **6** in
Load duration	**Ten years**

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **30.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **27.73** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **27.73** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **76.26** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **76.26** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**

Project	Job Ref.
I-10 Rest Areas	C190988.00

Tedds
GAI Consultants
600 Cranberry Woods Drive
Suite 400
Cranberry Township, PA. 16066

Section				Sheet no./rev.	
Porch Wood Column, LC7, 0.6DL+0.6WL				2	
Calc. by	Date	Chk'd by	Date	App'd by	Date
EJM	5/19/2020				

Temperature factor - Table 2.3.3 C_t = **1.00**

Size factor for bending - Table 4D C_{Fb} = **1.00**

Size factor for tension - Table 4D C_{Ft} = **1.00**

Size factor for compression - Table 4D C_{Fc} = **1.00**

Flat use factor - Table 4D C_{fu} = **1.00**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

Incising factor for bending, shear, tension & compression - Table 4.3.8

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Wet service factor for bending - Table 4D C_{Mb} = **1.00**

Wet service factor for tension - Table 4D C_{Mt} = **1.00**

Wet service factor for compression - Table 4D C_{Mc} = **0.91**

Wet service factor for perpendicular compression - Table 4D

C_{Mc_perp} = **0.67**

Wet service factor for shear - Table 4D C_{Mv} = **1.00**

Wet service factor for modulus of elasticity - Table 4D

C_{ME} = **1.00**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**

- Beam is fully restrained

Beam stability factor - cl.3.3.3 C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_{Mc\_perp} \times C_t \times C_i \times C_b$ = **251** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R / (N \times b \times L_b)$ = **10** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.041**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **850** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **467** lb/in^2

f_b / F_b' = **0.550**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i$ = **165** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A)$ = **7** lb/in^2

f_v / F_v' = **0.041**

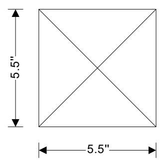
STRUCTURAL WOOD MEMBER DESIGN (NDS)

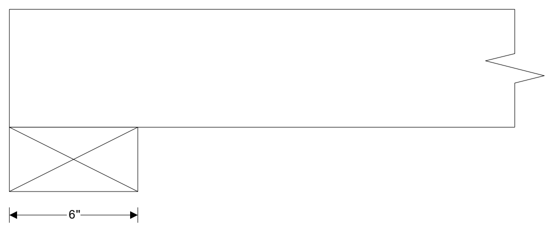
In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

Tedds calculation version 1.7.09

Analysis results

Design moment in major axis	M_x = **1022** lb_ft
Design shear	F = **99** lb
Maximum reaction	R = **3363** lb





Sawn lumber section details

Nominal breadth of sections	b_{nom} = **6** in
Dressed breadth of sections	b = **5.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **5.5** in
Species, grade and size classification	Southern Pine, No.2 grade, 5" x 5" and larger
Bending parallel to grain	F_b = **850** lb/in^2
Tension parallel to grain	F_t = **550** lb/in^2
Compression parallel to grain	F_c = **525** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **375** lb/in^2
Shear parallel to grain	F_v = **165** lb/in^2
Modulus of elasticity	E = **1080000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **440000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **67500** lb/in^2

Member details

Service condition	**Wet**
Length of bearing	L_b = **6** in
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **8** ft
Effective length factor in x-axis	K_x = **2.1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **16.8** ft
Unbraced length in y-axis	L_y = **10.333** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **10.333** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **30.25** in^2

Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{27.73}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{27.73}$ in^3

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{76.26}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{76.26}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.00}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4D $C_{Fb} = \mathbf{1.00}$

Size factor for tension - Table 4D $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4D $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4D $C_{fu} = \mathbf{1.00}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Wet service factor for bending - Table 4D $C_{Mb} = \mathbf{1.00}$

Wet service factor for tension - Table 4D $C_{Mt} = \mathbf{1.00}$

Wet service factor for compression - Table 4D $C_{Mc} = \mathbf{0.91}$

Wet service factor for perpendicular compression - Table 4D

$C_{Mc\_perp} = \mathbf{0.67}$

Wet service factor for shear - Table 4D $C_{Mv} = \mathbf{1.00}$

Wet service factor for modulus of elasticity - Table 4D

$C_{ME} = \mathbf{1.00}$

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} = \mathbf{440000}$ lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i = \mathbf{478}$ lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = \mathbf{269}$ lb/in^2

$c = \mathbf{0.80}$

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.48}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{1.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_{Mc\_perp} \times C_t \times C_{ic\_perp} \times C_b = \mathbf{251}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R / (N \times b \times L_b) = \mathbf{102}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.406}$

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_{Mb} \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{850}$ lb/in^2

Actual bending stress $f_b = M_x / S_x =$ **442** lb/in^2

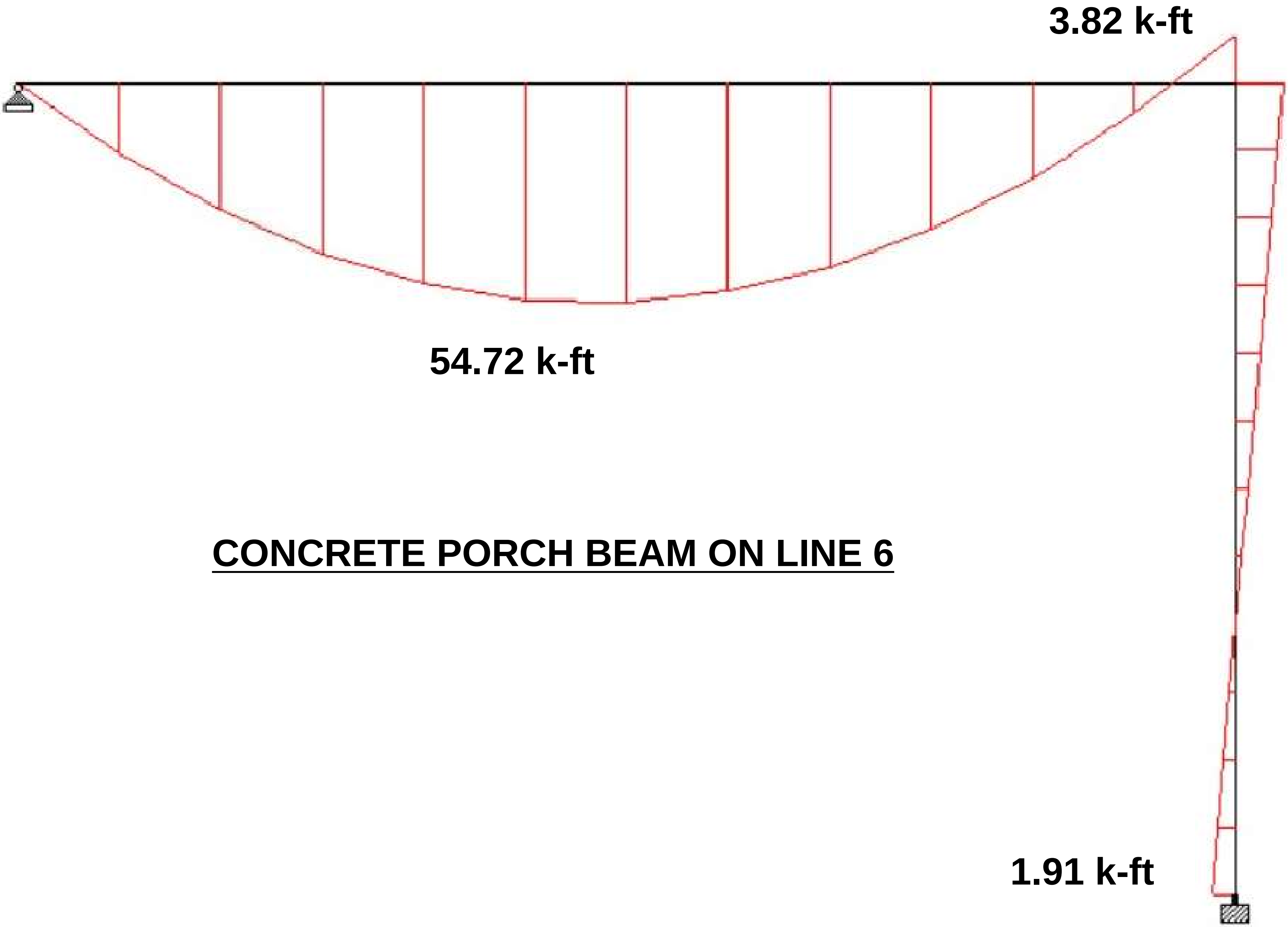
$f_b / F_b' =$ **0.520**

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_{Mv} \times C_t \times C_i =$ **165** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **5** lb/in^2

$f_v / F_v' =$ **0.030**



CONCRETE PORCH BEAM ON LINE 6

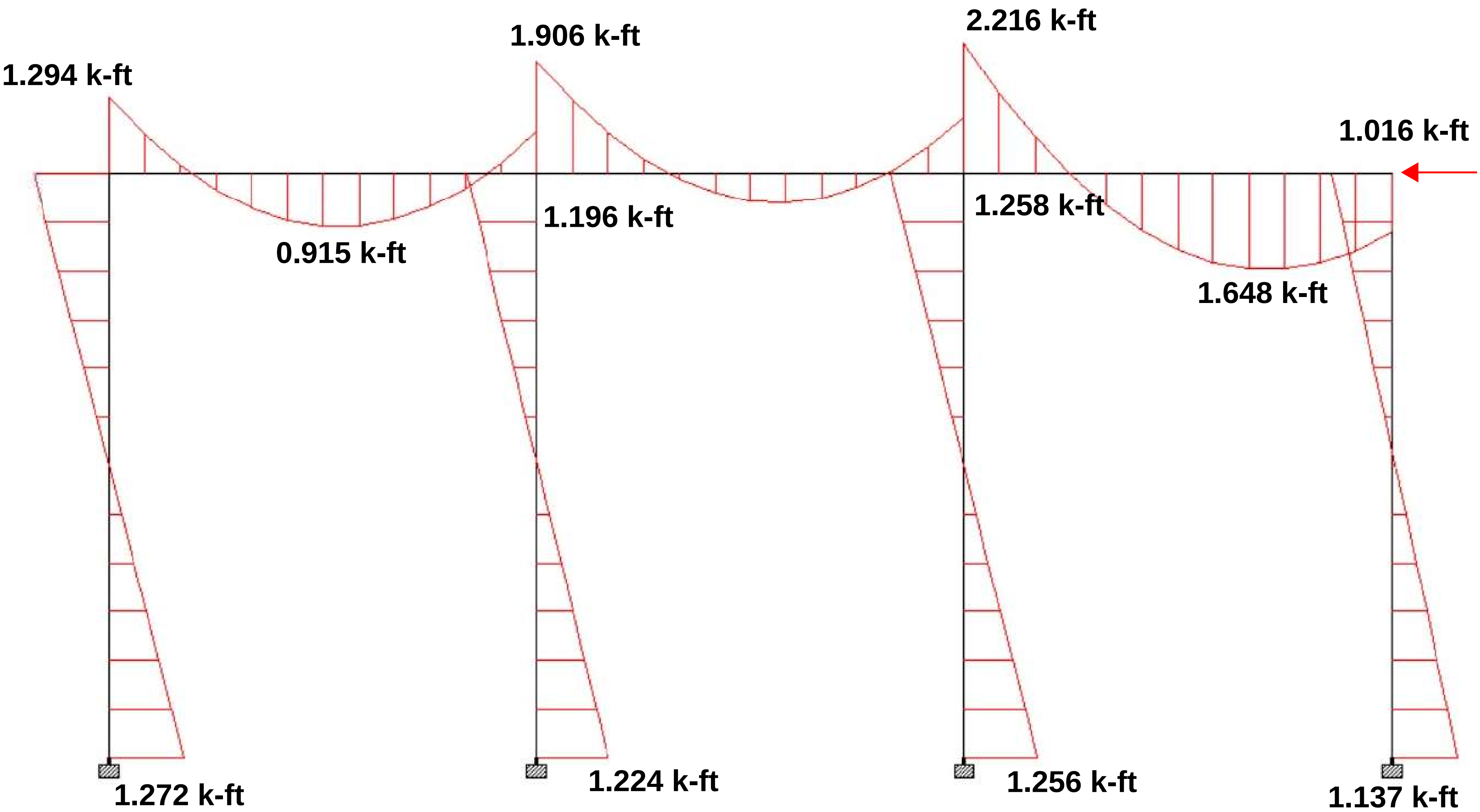
SUPPORT REACTIONS -UNIT POUN FEET STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	75.74	1970.81	0.00	0.00	0.00	0.00
	2	135.52	3510.15	0.00	0.00	0.00	0.00
	3	218.20	5651.40	0.00	0.00	0.00	0.00
	4	-176.02	-4559.14	0.00	0.00	0.00	0.00
	5	98.22	2544.03	0.00	0.00	0.00	0.00
	101	105.65	2745.47	0.00	0.00	0.00	0.00
	102	270.20	7007.38	0.00	0.00	0.00	0.00
	103	295.70	7667.89	0.00	0.00	0.00	0.00
	104	419.11	10864.32	0.00	0.00	0.00	0.00
	105	21.14	553.09	0.00	0.00	0.00	0.00
	106	185.69	4814.99	0.00	0.00	0.00	0.00
	201	362.61	9402.85	0.00	0.00	0.00	0.00
	202	651.74	16891.40	0.00	0.00	0.00	0.00
	203	514.62	13339.81	0.00	0.00	0.00	0.00
	204	460.84	11946.88	0.00	0.00	0.00	0.00
	205	186.59	4843.70	0.00	0.00	0.00	0.00
	206	288.36	7476.89	0.00	0.00	0.00	0.00
	207	14.11	373.72	0.00	0.00	0.00	0.00
3	1	-75.74	2929.01	0.00	0.00	0.00	336.40
	2	-135.52	3630.65	0.00	0.00	0.00	601.94
	3	-218.20	5845.40	0.00	0.00	0.00	969.13
	4	176.02	-4715.66	0.00	0.00	0.00	-781.82
	5	-98.22	2631.37	0.00	0.00	0.00	436.26
	101	-105.65	3730.27	0.00	0.00	0.00	469.25
	102	-270.20	8138.48	0.00	0.00	0.00	1200.10
	103	-295.70	8821.67	0.00	0.00	0.00	1313.36
	104	-419.11	12127.83	0.00	0.00	0.00	1861.50
	105	-21.14	1106.40	0.00	0.00	0.00	93.91
	106	-185.69	5514.62	0.00	0.00	0.00	824.76
	201	-362.61	10794.29	0.00	0.00	0.00	1610.57
	202	-651.74	18539.92	0.00	0.00	0.00	2894.74
	203	-514.62	14866.41	0.00	0.00	0.00	2285.70
	204	-460.84	13425.66	0.00	0.00	0.00	2046.83
	205	-186.59	6078.64	0.00	0.00	0.00	828.75
	206	-288.36	8535.06	0.00	0.00	0.00	1280.77
	207	-14.11	1188.04	0.00	0.00	0.00	62.68

************** END OF LATEST ANALYSIS RESULT **************

135. FINISH

LOAD CASE 201 0.9DL+1.0WL



SUPPORT REACTIONS -UNIT POUN FEET STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	13.29	1429.53	0.00	0.00	0.00	-59.01
	2	12.83	325.29	0.00	0.00	0.00	-56.99
	3	707.00	782.17	0.00	0.00	0.00	-4812.34
	4	176.52	195.29	0.00	0.00	0.00	-1201.51
	100	26.12	1754.82	0.00	0.00	0.00	-115.99
	101	450.32	2224.12	0.00	0.00	0.00	-3003.40
	102	132.03	1871.99	0.00	0.00	0.00	-836.90
	103	439.87	1522.19	0.00	0.00	0.00	-2957.00
	104	121.58	1170.06	0.00	0.00	0.00	-790.50
	200	18.60	2001.34	0.00	0.00	0.00	-82.61
	201	207.86	2301.07	0.00	0.00	0.00	-1340.70
	202	738.34	2887.95	0.00	0.00	0.00	-4951.53
	203	200.03	1774.62	0.00	0.00	0.00	-1305.90
	204	730.50	2361.51	0.00	0.00	0.00	-4916.73
3	1	-2.77	2306.55	0.00	0.00	0.00	12.33
	2	16.91	4060.99	0.00	0.00	0.00	-75.14
	3	740.59	-392.72	0.00	0.00	0.00	-4963.06
	4	184.90	-98.05	0.00	0.00	0.00	-1239.14
	100	14.14	6367.55	0.00	0.00	0.00	-62.81
	101	458.49	6131.92	0.00	0.00	0.00	-3040.65
	102	125.08	6308.72	0.00	0.00	0.00	-806.30
	103	452.84	3584.90	0.00	0.00	0.00	-3015.52
	104	119.43	3761.70	0.00	0.00	0.00	-781.17
	200	-3.88	3229.17	0.00	0.00	0.00	17.26
	201	201.87	7543.01	0.00	0.00	0.00	-1314.51
	202	757.56	7248.34	0.00	0.00	0.00	-5038.43
	203	197.63	5632.74	0.00	0.00	0.00	-1295.67
	204	753.31	5338.07	0.00	0.00	0.00	-5019.59
5	1	2.77	2306.55	0.00	0.00	0.00	-12.33
	2	-16.91	4060.99	0.00	0.00	0.00	75.14
	3	741.93	396.19	0.00	0.00	0.00	-4971.96
	4	185.24	98.92	0.00	0.00	0.00	-1241.36
	100	-14.14	6367.55	0.00	0.00	0.00	62.81
	101	431.01	6605.26	0.00	0.00	0.00	-2920.36
	102	97.00	6426.90	0.00	0.00	0.00	-682.00
	103	436.67	4058.24	0.00	0.00	0.00	-2945.49
	104	102.66	3879.88	0.00	0.00	0.00	-707.13
	200	3.88	3229.17	0.00	0.00	0.00	-17.26
	201	168.27	7739.97	0.00	0.00	0.00	-1165.99
	202	724.96	8037.25	0.00	0.00	0.00	-4896.58
	203	172.51	5829.71	0.00	0.00	0.00	-1184.83
	204	729.20	6126.98	0.00	0.00	0.00	-4915.43
7	1	-13.29	1429.53	0.00	0.00	0.00	59.01
	2	-12.83	325.29	0.00	0.00	0.00	56.99
	3	710.69	-785.65	0.00	0.00	0.00	-4837.59
	4	177.44	-196.15	0.00	0.00	0.00	-1207.81

SUPPORT REACTIONS -UNIT POUN FEET STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
	100	-26.12	1754.82	0.00	0.00	0.00	115.99
	101	400.30	1283.43	0.00	0.00	0.00	-2786.56
	102	80.34	1637.13	0.00	0.00	0.00	-608.70
	103	410.74	581.50	0.00	0.00	0.00	-2832.96
	104	90.79	935.20	0.00	0.00	0.00	-655.09
	200	-18.60	2001.34	0.00	0.00	0.00	82.61
	201	146.10	1909.63	0.00	0.00	0.00	-1068.62
	202	679.35	1320.14	0.00	0.00	0.00	-4698.40
	203	153.93	1383.18	0.00	0.00	0.00	-1103.42
	204	687.18	793.69	0.00	0.00	0.00	-4733.20

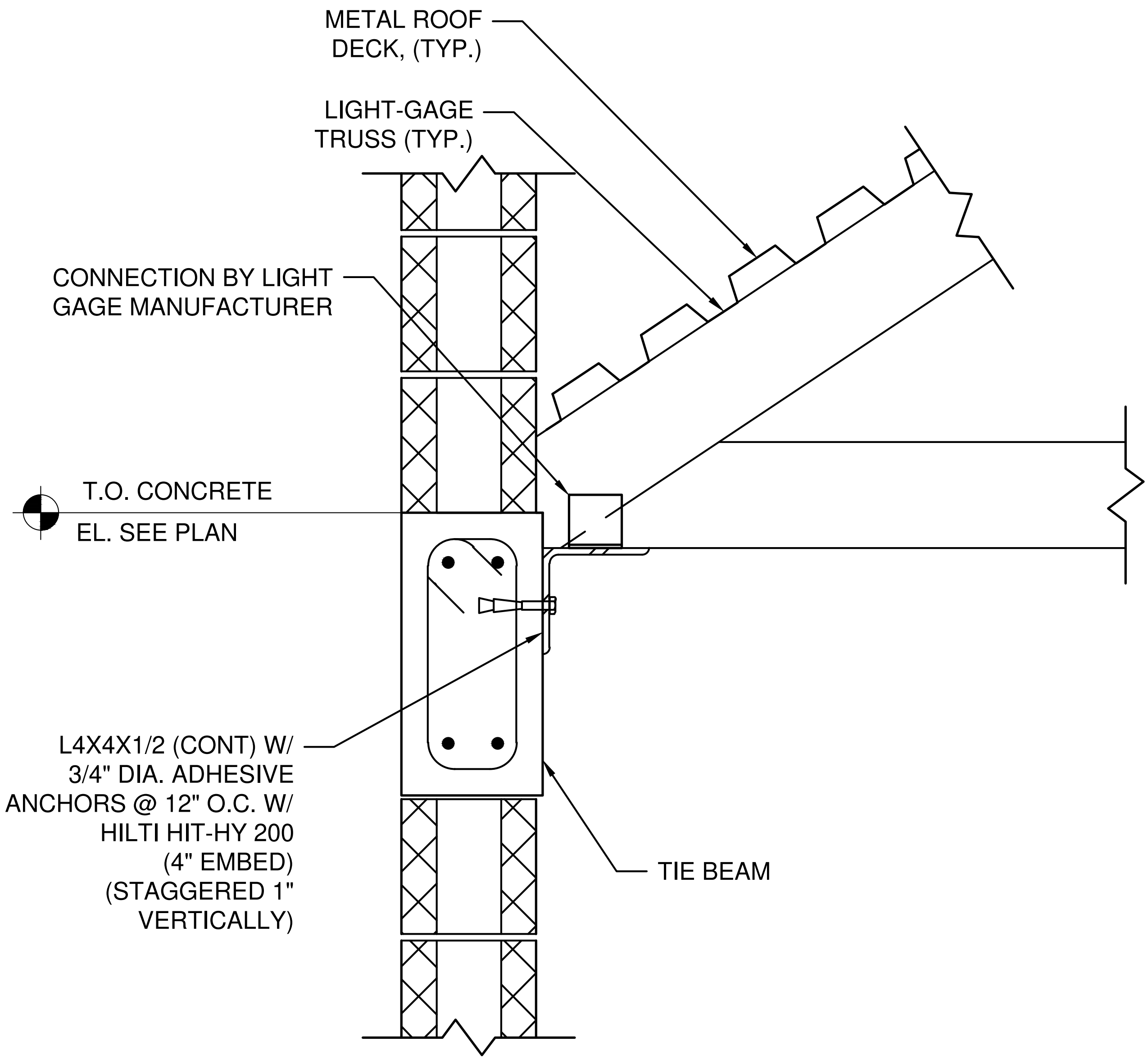
************** END OF LATEST ANALYSIS RESULT **************

119. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 22,2020 TIME= 15:51: 4 ****

Rest Room Building Connections


METAL ROOF
DECK, (TYP.)
LIGHT-GAGE
TRUSS (TYP.)
CONNECTION BY LIGHT
GAGE MANUFACTURER
T.O. CONCRETE
EL. SEE PLAN
L4X4X1/2 (CONT) W/
3/4" DIA. ADHESIVE
ANCHORS @ 12" O.C. W/
HILTI HIT-HY 200
(4" EMBED)
(STAGGERED 1"
VERTICALLY)
TIE BEAM

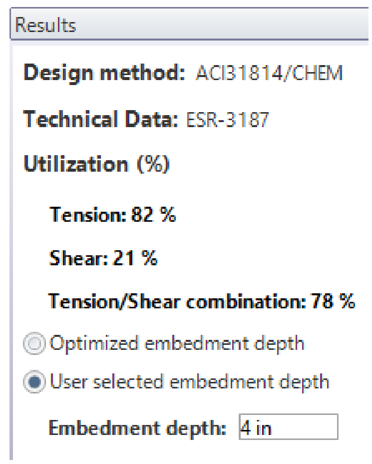
Company:	GAI	Page:	1
Specifier:	NJA	Project:	I-10 REst Area
Address:		Sub-Project I Pos. No.:	B190988.00
Phone I Fax:	|	Date:	9/4/2020
E-Mail:			

Specifier's comments: Roof Truss wall anchors

Anchor	Size (depth)	Total	Geometry	Embedment d
HIT-HY 200 + HAS-E	3/4	82 %		4

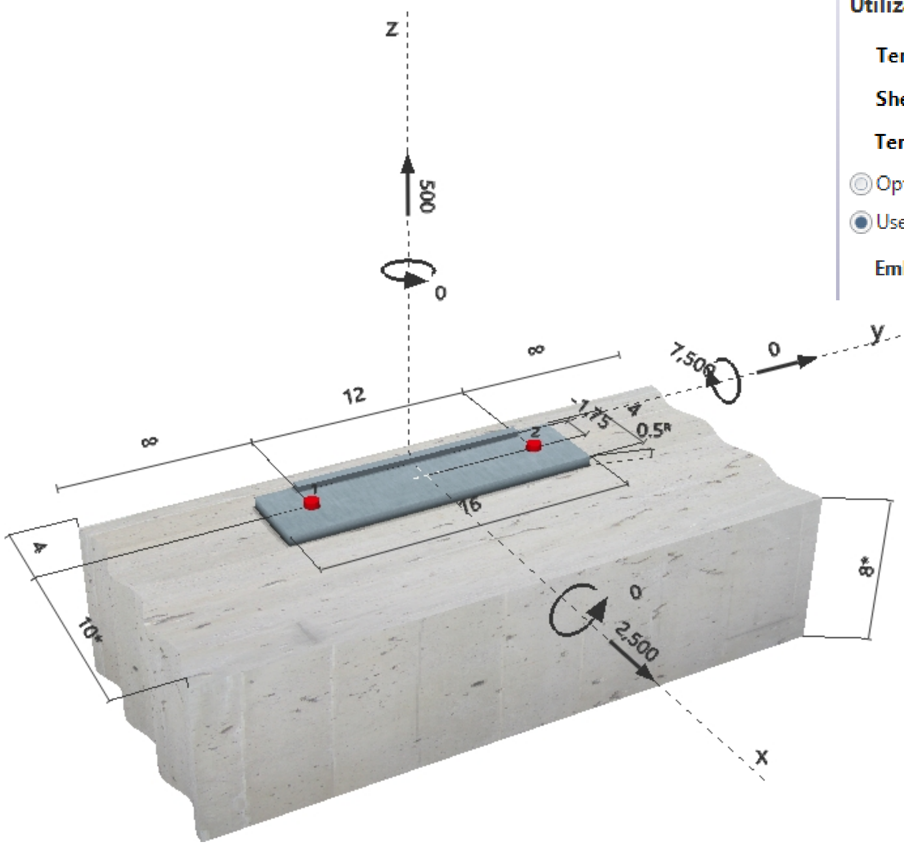
1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-E 3/4**
Return period (service life in years):	50
Effective embedment depth:	$h_{ef,act}$ = 4.000 in. ($h_{ef,limit}$ = - in.)
Material:	5.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	4/1/2019 | 3/1/2020
Proof:	Design method ACI 318-14 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 4.000 in. x 16.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Rectangular plates and bars (AISC), ; (L x W x T) = 0.500 in. x 12.000 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar



R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



Company:	GAI	Page:	2
Specifier:	NJA	Project:	I-10 REst Area
Address:		Sub-Project I Pos. No.:	B190988.00
Phone I Fax:	|	Date:	9/4/2020
E-Mail:			

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb] Load	Capacity	Utilization N / V [%]	Status
Tension	Bond Strength	5,317	6,563	82 / -	OK
Shear	Concrete edge failure in direction x+	2,500	12,128	- / 21	OK

Loading	N	V		Utilization N,V [%]	Status
Combined tension and shear loads	0.810	0.206	5/3	78	OK

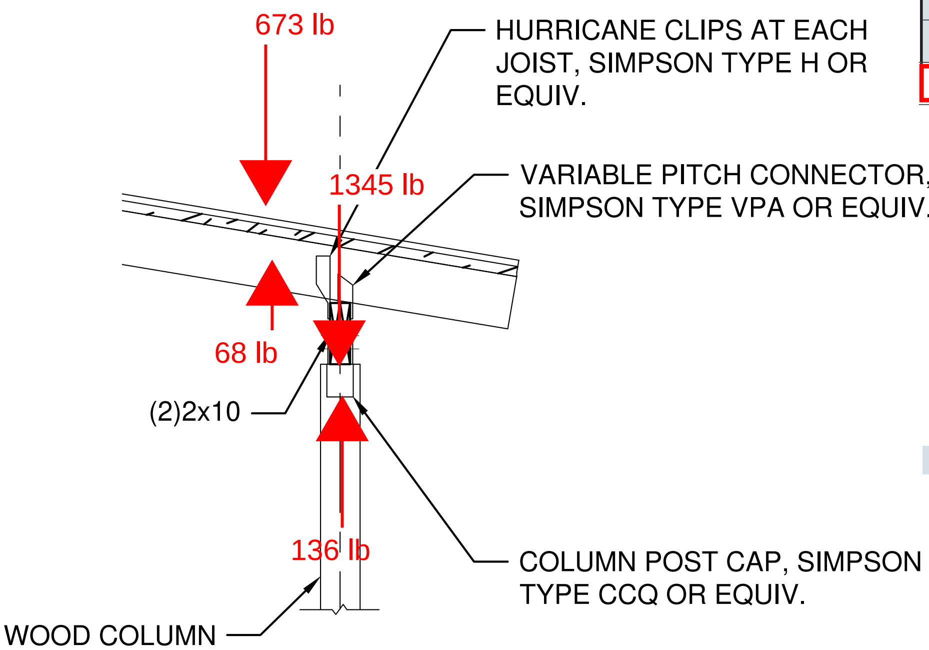
3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

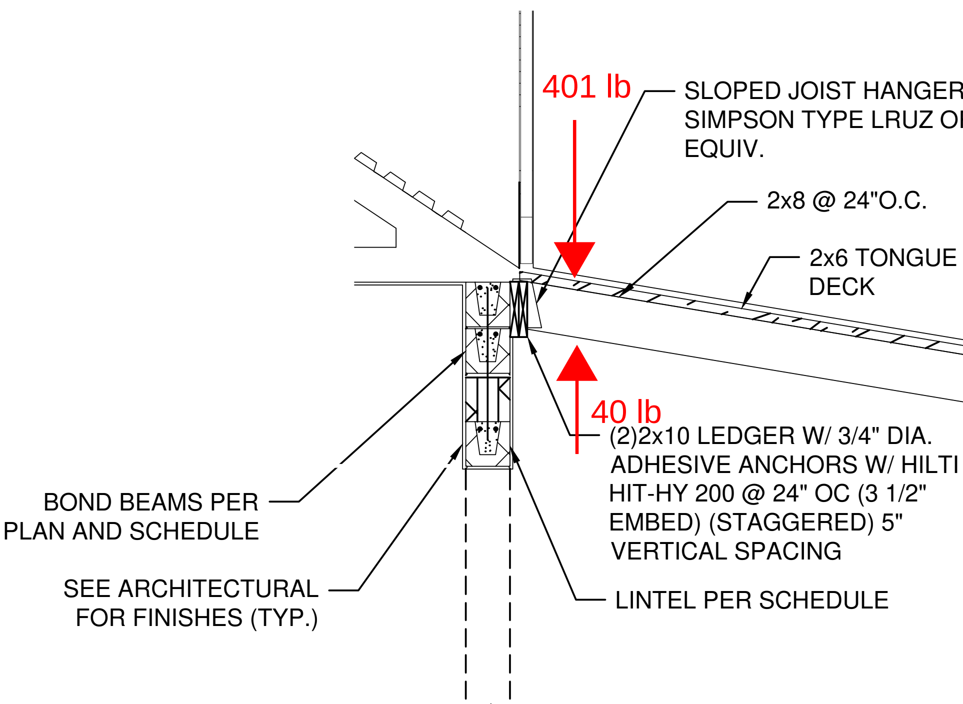
- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.



DF/SP Allowable Loads
Uplift
(160)
565

DF/SP Allowable Loads			
Uplift	Download	Lateral	
		(160)	
(160)	(100/115/125)	F_1	F_2
255	1,105	345	300

Allowable Loads (DF/SP)	
CCQ	
Uplift	Down
(160)	(100)
6,785	24,065



DF/SP Allowable Loads			
Uplift (160)	Floor (100)	Snow (115)	Roof (125)
810	1,030	1,175	1,275

Shear — ΦV_n
f'_c = 3,000 psi (20.7 MPa) lb (kN)
12,050 (53.6)

CCQ/ECCQ

COLUMN (6X6) TO BEAM (2X10) CONNECTION
MAX DOWNWARD LOAD: 1345 LB
MAX UPLIFT: 136 LB

Column Caps

This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

Column caps provide a strong connection for column-beam combinations. This design uses Strong-Drive® SDS Heavy-Duty Connector screws to provide faster installation and provides a greater net section area of the column compared to bolts. The SDS screws provide for a lower profile compared to standard through bolts.

Material: CCQ3, ECCQ3, CCQ4, CCQ4.62, ECCQ4, ECCQ4.62, CCQ6, ECCQ6 — 7 gauge; all others — 3 gauge

Finish: Simpson Strong-Tie gray paint; available in HDG and stainless steel; CCOQ and ECCOQ — no coating

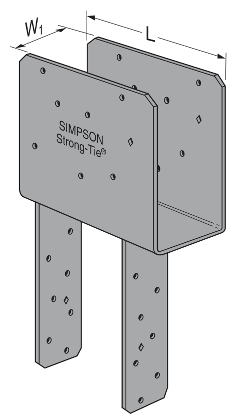
Installation:

- Install ¼" x 2½" Strong-Drive SDS Heavy-Duty Connector screws, which are provided with the column cap. (Lag screws will not achieve the same load.) Install stainless-steel Strong-Drive screws with stainless-steel connectors.
- CCOQ and ECCOQ column caps only (no straps) may be ordered for field-welding to pipe or other columns. Dimensions are same as CCQ and ECCQ. Weld by Designer.
- For rough-cut lumber sizes, provide dimensions. An optional W_2 dimension may be specified with any column size given. (Note that the W_2 dimension on straps rotated 90° is limited by the W_1 dimension.)

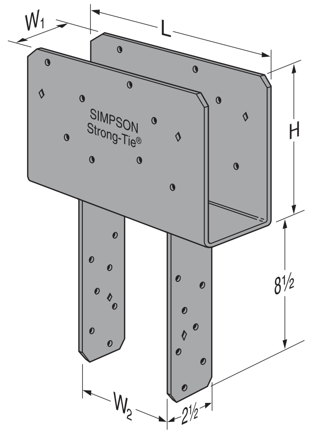
Options:

- For end conditions, specify ECCQ.
- Straps may be rotated 90° where $W_1 \geq W_2$ and for CCQ5-6.
- Other custom column caps are available. Contact Simpson Strong-Tie.

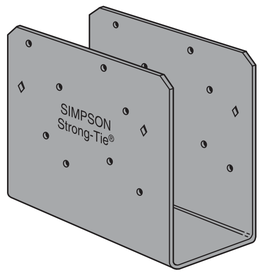
Codes: See p. 12 for Code Reference Key Chart



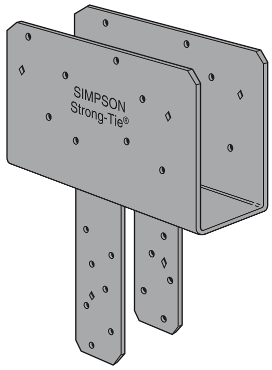
ECCQ46SDS2.5



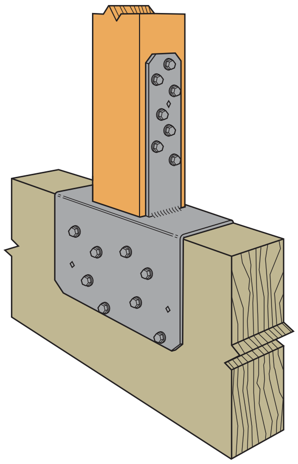
CCQ46SDS2.5



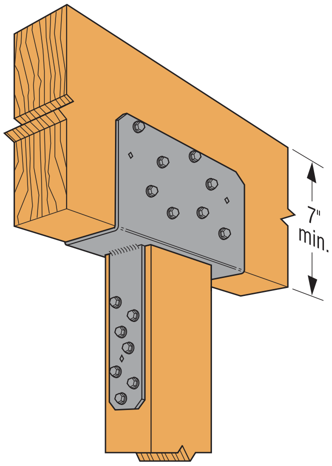
CCOQ4-SDS2.5
(no coating)



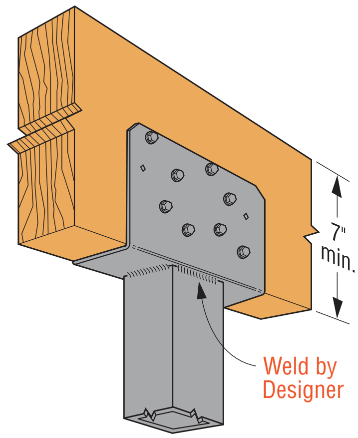
Optional CCQ with Straps Rotated 90°



Inverted CCQ44SDS2.5 Post-to-Beam Installation



Typical CCQ46SDS2.5 Installation



CCOQ Installation on Steel Column

SIMPSON Strong-Tie®

Column Caps (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

Model No.	Beam Width (in.)	Dimensions (in.)					No. of ¼" x 2½" SDS Screws			Allowable Loads (DF/SP)				Code Ref.	CCOQ/ECCOQ Model No. (No Legs)
		W_1	W_2	L		H	Beam		Post	CCQ		ECCQ			
				CCQ	ECCQ		CCQ	ECCQ		Uplift (160)	Down (100)	Uplift (160)	Down (100)		
CCQ3-4SDS2.5	3⅛	3¼	3⅝	11	8½	7	16	14	14	5,370	16,980	3,465	6,125	IBC, FL, LA	CCOQ3-SDS2.5 ECCOQ3-SDS2.5
CCQ3-6SDS2.5	3⅛	3¼	5½	11	8½	7	16	14	14	5,370	21,485	3,465	10,740		
CCQ44SDS2.5	3½	3⅝	3⅝	11	8½	7	16	14	14	5,370	19,020	3,785	7,655		CCOQ4-SDS2.5 ECCOQ4-SDS2.5
CCQ46SDS2.5	3½	3⅝	5½	11	8½	7	16	14	14	6,785	24,065	3,785	12,030		
CCQ48SDS2.5	3½	3⅝	7½	11	8½	7	16	14	14	6,785	24,065	3,785	16,405		
CCQ4.62-3.62SDS	4½	4⅝	3⅝	11	8½	7	16	14	14	5,370	23,390	3,785	9,845		CCOQ4.62-SDS2.5 ECCOQ4.62-SDS2.5
CCQ4.62-4.62SDS	4½	4⅝	4⅝	11	8½	7	16	14	14	5,370	30,070	3,785	12,655		
CCQ4.62-5.50SDS	4½	4⅝	5½	11	8½	7	16	14	14	6,785	30,940	3,785	15,470		
CCQ5-4SDS2.5	5⅛	5¼	3⅝	11	8½	7	16	14	14	5,370	26,635	4,040	11,210		CCOQ5-SDS2.5 ECCOQ5-SDS2.5
CCQ5-6SDS2.5	5⅛	5¼	5½	11	8½	7	16	14	14	6,785	28,190	5,355	17,615		
CCQ5-8SDS2.5	5⅛	5¼	7½	11	8½	7	16	14	14	6,785	35,235	5,355	24,025		
CCQ64SDS2.5	5¼, 5½	5½	3⅝	11	8½	7	16	14	14	5,370	28,585	3,785	12,030		CCOQ6-SDS2.5 ECCOQ6-SDS2.5
CCQ66SDS2.5	5¼, 5½	5½	5½	11	8½	7	16	14	14	6,785	33,275	3,785	18,905		
CCQ68SDS2.5	5¼, 5½	5½	7½	11	8½	7	16	14	14	6,785	37,815	3,785	25,780		
CCQ6-7.13SDS2.5	5¼, 5½	5½	7⅛	11	8½	7	16	14	14	6,785	37,815	3,785	24,490		
CCQ74SDS2.5	6¾	6⅞	3⅝	11	8½	7	16	14	14	5,370	33,490	4,040	15,355		CCOQ7-SDS2.5 ECCOQ7-SDS2.5
CCQ76SDS2.5	6¾	6⅞	5½	11	8½	7	16	14	14	6,785	37,125	5,355	24,130		
CCQ77SDS2.5	6¾	6⅞	6⅞	11	8½	7	16	14	14	6,785	48,265	5,355	29,615		
CCQ78SDS2.5	6¾	6⅞	7½	11	8½	7	16	14	14	6,785	48,265	5,355	32,905		
CCQ7.1-4SDS2.5	7	7⅛	3⅝	11	8½	7	16	14	14	5,370	34,730	4,040	18,375		CCOQ7.12-SDS2.5 ECCOQ7.12-SDS2.5
CCQ7.1-6SDS2.5	7	7⅛	5½	11	8½	7	16	14	14	6,785	38,500	5,355	28,875		
CCQ7.1-7.1SDS2.5	7	7⅛	7⅛	11	8½	7	16	14	14	6,785	57,750	5,355	36,750		
CCQ7.1-8SDS2.5	7	7⅛	7½	11	8½	7	16	14	14	6,785	52,500	5,355	39,375		
CCQ84SDS2.5	7½	7½	3⅝	11	8½	7	16	14	14	6,785	37,210	5,355	16,405		CCOQ8-SDS2.5 ECCOQ8-SDS2.5
CCQ86SDS2.5	7½	7½	5½	11	8½	7	16	14	14	6,785	41,250	5,355	25,780		
CCQ88SDS2.5	7½	7½	7½	11	8½	7	16	14	14	6,785	51,565	5,355	35,155		
CCQ94SDS2.5	8¾	8⅞	3⅝	11	8½	7	16	14	14	6,785	47,545	5,355	19,905		CCOQ9-SDS2.5 ECCOQ9-SDS2.5
CCQ96SDS2.5	8¾	8⅞	5½	11	8½	7	16	14	14	6,785	48,125	5,355	31,280		
CCQ98SDS2.5	8¾	8⅞	7½	11	8½	7	16	14	14	6,785	62,565	5,355	42,655		
CCQ106SDS2.5	9¼	9½	5½	11	8½	7	16	14	14	6,785	52,250	5,355	32,655		CCOQ10-SDS2.5 ECCOQ10-SDS2.5

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed. Reduce where other loads govern.
2. Downloads shall be reduced where limited by capacity of the post.
3. Uplift loads do not apply to spliced conditions. Spliced conditions must be detailed by the Designer to transfer tension loads between spliced members by means other than the post cap.
4. Spliced conditions must be detailed by the Designer to transfer tension loads between spliced members by means other than the column cap.
5. Column sides are assumed to be aligned in the same vertical plane as the beam sides. CCQ4.62 models assume a minimum 3½"-wide post.
6. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers known as the narrow face. Values in the tables reflect installation into the wide face. See technical bulletin T-C-SCLCLM at **strongtie.com** for load reductions resulting from narrow-face installations.
7. Beam depth must be a minimum of 7".
8. For 5¼" engineered lumber, use 5½" models.
9. CCOQ and ECCOQ welded to a steel column will achieve maximum load listed as CCQ and ECCQ. The steel column width shall match the beam width. Weld by Designer.

Bases and Caps

SIMPSON Strong-Tie®

VPA

BEAM (2X10) TO RAFTER (2X8) CONNECTION
MAXIMUM DOWNWARD LOAD: 673 LB
MAXIMUM UPLIFT: 68 LB
LATERAL LOAD: 0 LB

Variable-Pitch Connector

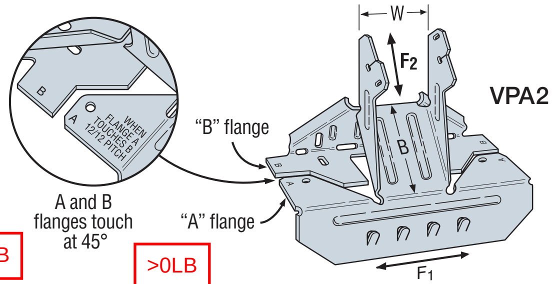
The VPA may be sloped in the field, offering a versatile solution for attaching rafters to the top plate. It will adjust to accommodate slopes between 3:12 and 12:12, making it a complement to the versatile LSSR and LSSU hangers. This connector eliminates the need for notched rafters, beveled top plates and toe nailing.

Material: 18 gauge

Finish: Galvanized

Installation: • Use all specified fasteners; see General Notes

Codes: See p. 12 for Code Reference Key Chart



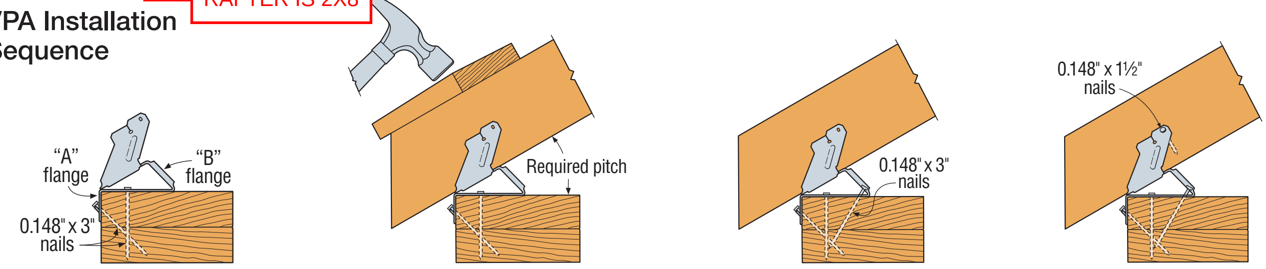
>68LB   >673 LB   >0LB

Joist Width	Model No.	W (in.)	Fasteners (in.) Carrying Member	Fasteners (in.) Carried Member	DF/SP Allowable Loads Uplift (160)	DF/SP Download (100/115/125)	DF/SP Lateral (160) F_1	DF/SP Lateral (160) F_2	SPF/HF Allowable Loads Uplift (160)	SPF/HF Download (100/115/125)	SPF/HF Lateral (160) F_1	SPF/HF Lateral (160) F_2	Code Ref.
1½	VPA2	1 9/16	(8) 0.148 x 3	(2) 0.148 x 1½	255	1,105	345	300	220	950	295	260	IBC, FL, LA
2½	VPA3	2 9/16	(9) 0.148 x 3	(2) 0.148 x 1½	255	1,245	345	300	220	1,070	295	260	
3½	VPA4	3 9/16	(11) 0.148 x 3	(2) 0.148 x 1½	255	1,245	345	300	220	1,070	295	260	

1. Loads have been increased for wind or earthquake loading, with no further increase allowed. Reduce where other loads govern.
2. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

RAFTER IS 2X8

VPA Installation Sequence



Step 1
Install top nails and face PAN nails in "A" flange to outside wall top plate.

Step 2
Seat rafter with a hammer, adjusting "B" flange to the required pitch.

Step 3
Install "B" flange nails in the obround nail holes, locking the pitch.

Step 4
Install 0.148" x 1½" nail into tab nail hole. Hammer nail in at a slight angle to prevent splitting.

HCP

Hip Corner Plate

The HCP connects a rafter or joist to double top plates at a 45° angle.

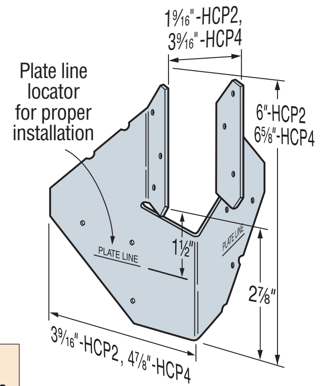
Material: 18 gauge

Finish: HCP2 — galvanized or ZMAX® coating; HCP4Z — ZMAX coating

Installation: • Use all specified fasteners; see General Notes.

- Attach HCP to double top plates; birdsmouth not required for table uplift loads but may be required for download.
- Install rafter and complete nailing. Rafter may be sloped to 45°.

Codes: See p. 12 for Code Reference Key Chart

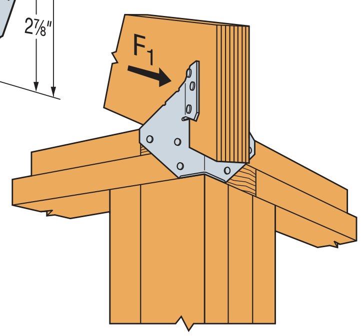


HCP2
(HCP4Z similar)
U.S. Patent 5,380,115

These products are available with additional corrosion protection. For more information, see p. 15.

Member Size	Model No.	Fasteners (in.) To Rafters	Fasteners (in.) To Plates	DF/SP Allowable Loads (160) Uplift	DF/SP Allowable Loads (160) F_1	SPF/HF Allowable Loads (160) Uplift	SPF/HF Allowable Loads (160) F_1	Code Ref.
2x	HCP2	(6) 0.148 x 1½	(6) 0.148 x 1½	590	255	510	220	IBC, FL, LA
4x	HCP4Z	(8) 0.148 x 3	(8) 0.148 x 3	990	230	850	200	

1. Loads have been increased for wind or earthquake loading, with no further increase allowed. Reduce where other loads govern.
2. The HCP can be installed on the inside and the outside of the wall with a flat bottom chord truss and achieve twice the allowable load.
3. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.



Typical HCP Installation

UPDATED 06/01/19

H/TSP

BEAM (2X10) TO RAFTER (2X8) CONNECTION
MAX UPLIFT: 68 LB

Seismic and Hurricane Ties

Simpson Strong-Tie hurricane ties provide a positive connection between truss/rafter and the wall of the structure to resist wind and seismic forces.

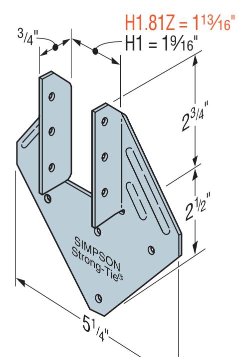
Material: See table

Finish: Galvanized. H1.81Z, H7Z and H11Z — ZMAX® coating. Some models available in stainless steel or ZMAX; see Corrosion Information, pp. 13–15 or visit **strongtie.com**.

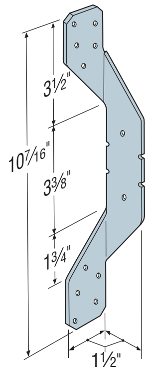
Installation:

- Use all specified fasteners; see General Notes.
- Hurricane ties can be installed with flanges facing inward or outward.
- H2.5T, H3 and H6 ties are shipped in equal quantities of right and left versions (right versions shown).
- Hurricane ties do not replace solid blocking.
- When installing ties on plated trusses (on the side opposite the truss plate) do not fasten through the truss plate from behind. This can force the truss plate off of the truss and compromise truss performance.
- H10A optional nailing to connect shear blocking, use 8d nails. Slots allow maximum field bending up to a pitch of 6:12, use H10A sloped loads for field-bent installation.

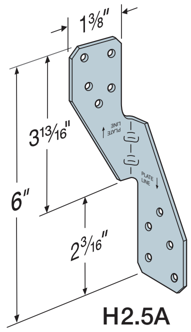
Codes: See p. 12 for Code Reference Key Chart



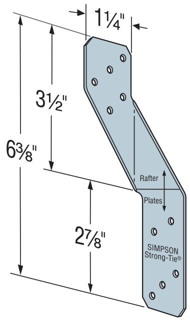
H1
NEW *(H1.81Z similar)*



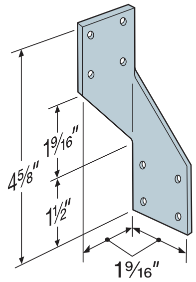
H2A
(H2ASS similar)



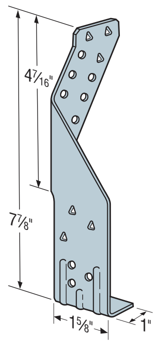
H2.5A
(H2.5ASS similar)



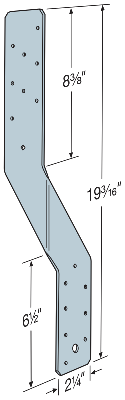
H2.5T



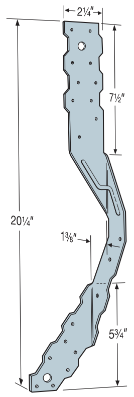
H3



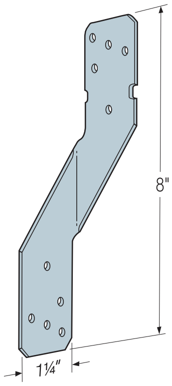
TSP



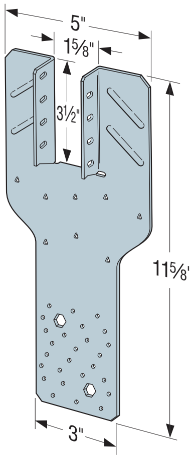
H6



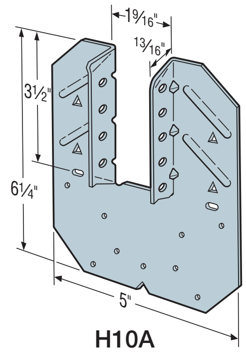
H7Z



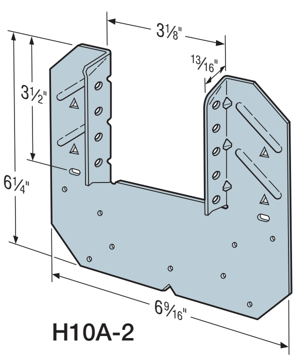
H8



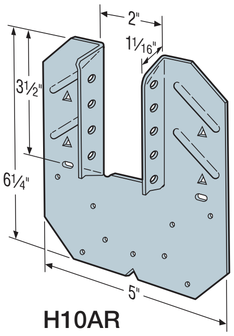
H10S



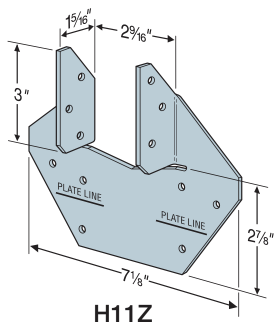
H10A
(H10ASS similar)



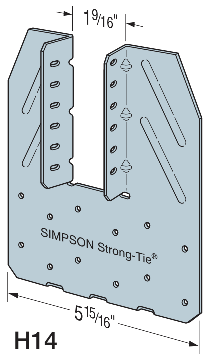
H10A-2



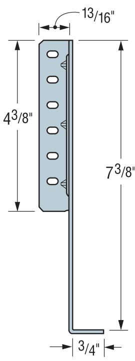
H10AR



H11Z



H14



H14 Profile

Straps and Ties

SIMPSON Strong-Tie®

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Model No.	Ga.	Fasteners (in.) To Rafters/ Truss	Fasteners (in.) To Plates	Fasteners (in.) To Studs	DF/SP Allowable Loads Uplift (160)	DF/SP Lateral (160) F_1	DF/SP Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads Uplift (160)	SPF/HF Lateral (160) F_1	SPF/HF Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	Code Ref.
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	350	335	195	330	300	290	150	260	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	565	110	110	575	535	110	110	495	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	495	135	145	420	495	135	145	420	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	415	365	180	145	290	
H6	16	—	(8) 0.131 x 2½	(8) 0.131 x 2½	1,230	—	—	—	1,055	—	—	—	IBC, FL
H7Z	16	(4) 0.131 x 2½	(2) 0.131 x 1½	(8) 0.131 x 2½	830	410	—	—	715	355	—	—	
H8	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	780	95	90	630	710	95	90	510	IBC, FL, LA
H10A Field Bent	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	855	590	285	—	760	505	285	—	
H10A	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,040	565	285	—	1,015	485	285	—	
H10ASS	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	970	565	170	—	835	485	170	—	—
H10AR	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,050	490	285	—	905	420	285	—	
H10S	18	(8) 0.131 x 1½	(8) 0.131 x 1½	(8) 0.131 x 2½	910	660	215	550	785	570	185	475	IBC, FL, LA
H10A-2	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,080	680	260	—	930	585	225	—	
H11Z	18	(6) 0.162 x 2½	(6) 0.162 x 2½	—	830	525	760	—	715	450	655	—	—
H14	18	(12) 0.131 x 1½	(13) 0.131 x 2½	—	1,275	725	285	—	1,050	480	245	—	IBC, FL, LA
		(12) 0.131 x 1½	(15) 0.131 x 2½	—	1,340	670	230	—	1,050	480	245	—	
TSP	16	(9) 0.148 x 1½	(6) 0.148 x 1½	—	755	310	190	—	650	265	160	—	FL
		(9) 0.148 x 1½	(6) 0.148 x 3	—	1,015	310	190	—	875	265	160	—	

1. See pp. 260–261 for Straps and Ties General Notes.
2. Allowable loads are for one anchor. A minimum rafter thickness of 2½" must be used when framing anchors are used on each side of the joist and on the same side of the plate (exception: connectors installed such that nails on opposite side don't interfere).
3. Allowable DF/SP uplift load for stud-to-bottom plate installation (see detail 15) is 390 lb. (H2.5A); 265 lb. (H2.5ASS); and 310 lb. (H8). For SPF/HF values, multiply these values by 0.86.
4. Allowable loads in the F_1 direction are not intended to replace diaphragm boundary members or cross-grain bending of the truss or rafter members.
5. When cross-grain bending or cross-grain tension cannot be avoided in the members, mechanical reinforcement to resist such forces shall be considered by the Designer.
6. Hurricane ties are shown on the outside of the wall for clarity and assume a minimum overhang of 3½". Installation on the inside of the wall is acceptable (see General Instructions for the Installer, note "s" on p. 18). For uplift continuous load path, install connectors in the same area (e.g., truss-to-plate connector and plate-to-stud connector) on the same side of the wall, unless detailed by designer. See technical bulletin T-C-HTIECON at **strongtie.com** for more information.
7. Southern pine allowable uplift loads for H10A = 1,340 lb. and for the H14 = 1,465 lb.
8. Refer to Simpson Strong-Tie® technical bulletin T-C-HTIEBEAR at **strongtie.com** for allowable bearing enhancement loads.
9. H10S can have the stud offset a maximum of 1" from the rafter (center to center) for a reduced uplift of 890 lb. (DF/SP) and 765 lb. (SPF).
10. H10S nails to plates are optional for uplift but required for lateral loads.
11. Some load values for the stainless-steel connectors shown here are lower than those for the carbon-steel versions. Ongoing test programs have shown this also to be the case with other stainless-steel connectors in the product line that are installed with nails. Visit **strongtie.com/corrosion** for updated information.
12. The allowable loads of stainless-steel connectors match carbon-steel connectors when installed with stainless-steel Strong-Drive® SCNR Ring-Shank Connector nails. For more information, refer to engineering letter L-F-SSNAILS at **strongtie.com**.
13. Allowable DF/SP/SPF uplift load for the H2.25A fastened to a 2x4 truss bottom chord and double top plates using (5) 0.131" x 1½" nails in the top plates and (3) 0.131" x 1½" nails in the lowest three flange holes into the truss bottom chord is 260 lb. (160).
14. For TSP installed stud to single plate see p. 276.
15. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

LRUZ

JOIST TO LEDGER CONNECTION
MAX DOWNWARD LOAD: 401 LB
MAX UPLIFT: 40 LB

Face-Mount Rafter Hanger

The LRUZ offers an economic alternative for those applications requiring a sloped hanger for rafter-to-ridge connections. Used with solid sawn rafters, the LRUZ's unique design enables the hanger to be installed either before or after the rafter is in place. The field-adjustable seat helps improve job efficiency by eliminating mismatched angles in the field and lead times associated with special orders. The LRUZ offers comparable or better load capacity to other rafter hangers at a reduced cost while using fewer fasteners.

Features:

- The open design and ability to field-adjust the slope make the LRUZ ideal for both retrofit or new applications.
- Accommodates roof pitches from 0:12 to 14:12.
- Slopes up or down to 45° (12:12). For downward slopes greater than 45° up to 49° (14:12), allowable downloads are 0.85 of table loads.
- For added versatility, the fasteners on the face of the hanger are placed high enabling the bottom of the rafter to hang below the ridge beam (see "Max. C_1" dimension).
- Can be installed using nails or Strong-Drive® SD Connector screws.

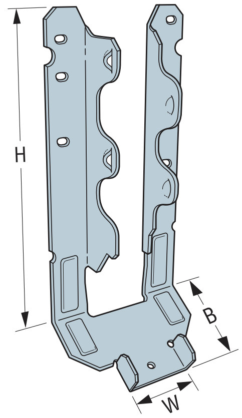
Material: 18 gauge

Finish: ZMAX® coating (G-185)

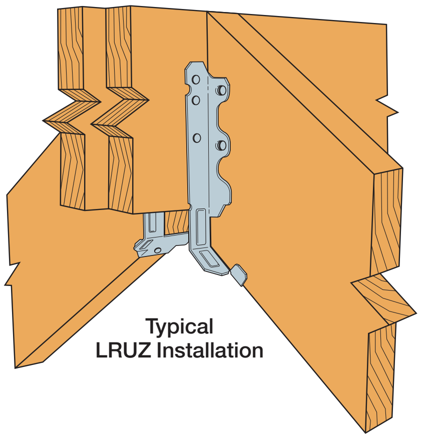
Installation:

- Use all specified fasteners; see General Notes
- Joist fasteners must be installed at an angle through the rafter or joist into the header to achieve the table loads
- See alternate installation on p. 115 for retrofit applications

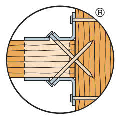
Codes: See p. 12 for Code Reference Key Chart



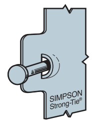
LRU28Z
(other models similar)



Typical LRUZ Installation



Double-Shear Nailing Top View



Dome Double-Shear Nailing Side View

These products are available with additional corrosion protection. For more information, see p. 15.

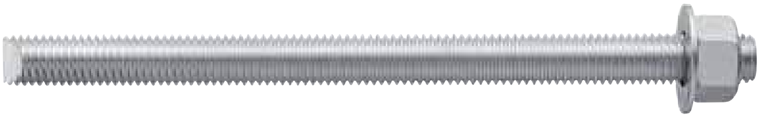
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Standard Installation

Model No.	Dimensions (in.)				Fasteners (in.)		DF/SP Allowable Loads				SPF/HF Allowable Loads				Code Ref.
	W	H	B	Max. C_1	Face	Joist	Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Uplift (160)	Floor (100)	Snow (115)	Roof (125)	
LRU26Z	1 9/16	5 1/4	1 15/16	1 3/4	(4) 0.162 x 3½	(5) 0.162 x 3½	810	1,030	1,175	1,275	695	885	1,010	1,095	IBC, FL
					(4) 0.148 x 3	(5) 0.148 x 3	600	865	990	990	515	745	850	850	
					(4) SD #10 x 2½	(5) SD #10 x 2½	770	1,215	1,395	1,425	660	935	1,075	1,170	
					(4) SD #10 x 1½	(5) SD #10 x 2½	770	1,045	1,200	1,305	660	830	950	1,035	
LRU28Z	1 9/16	6 15/16	1 15/16	2 5/8	(6) 0.162 x 3½	(5) 0.162 x 3½	810	1,315	1,340	1,340	695	1,130	1,150	1,150	
					(6) 0.148 x 3	(5) 0.148 x 3	805	1,050	1,050	1,050	690	905	905	905	
					(6) SD #10 x 2½	(5) SD #10 x 2½	1,025	1,480	1,480	1,480	880	1,265	1,270	1,270	
					(6) SD #10 x 1½	(5) SD #10 x 2½	1,025	1,390	1,480	1,480	880	1,105	1,270	1,270	
LRU210Z	1 9/16	8 3/16	1 15/16	1 3/4	(6) 0.162 x 3½	(7) 0.162 x 3½	1,015	1,550	1,620	1,620	875	1,335	1,395	1,395	
					(6) 0.148 x 3	(7) 0.148 x 3	1,015	1,295	1,480	1,495	875	1,115	1,275	1,285	
					(6) SD #10 x 2½	(7) SD #10 x 2½	1,510	1,805	1,805	1,805	1,300	1,405	1,550	1,550	
					(6) SD #10 x 1½	(7) SD #10 x 2½	1,510	1,570	1,805	1,805	1,300	1,240	1,430	1,550	
LRU212Z	1 9/16	10 11/16	1 15/16	3 1/2	(6) 0.162 x 3½	(7) 0.162 x 3½	1,305	1,550	1,765	1,910	1,120	1,335	1,520	1,645	
					(6) 0.148 x 3	(7) 0.148 x 3	1,305	1,295	1,430	1,430	1120	1,115	1,230	1,230	
					(6) SD #10 x 2½	(7) SD #10 x 2½	1,850	1,820	1,915	1,915	1,590	1,405	1,615	1,645	
					(6) SD #10 x 1½	(7) SD #10 x 2½	1,850	1,570	1,805	1,915	1,590	1,240	1,430	1,555	

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed. Reduce where other loads govern.
2. Allowable loads are based on a minimum 3" carrying member. For single 2x carrying members, use 0.148" x 1½" nails in the face and 0.148" x 3" in the joist, and reduce the allowable load to 0.81 of the tabulated value for 0.148" x 3" nails. Alternatively, use #10 x 1½" Strong-Drive® SD Connector screws in the face and #10 x 2½" SD Connector screws in the joist as shown in the table.
3. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.
4. **Fasteners:** SD screws are Simpson Strong-Tie® Strong-Drive® screws. See pp. 335–337 for fastener information.

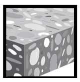
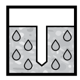
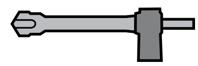
HIT-HY 200 Adhesive with HAS Threaded Rod



LEDGER CONNECTION TO CMU BOND BEAM - ANCHORS
MAX SHEAR: 401 LB

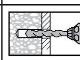
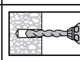
Hilti HAS threaded rod

Figure 9 - Hilti HAS threaded rod installation conditions

Permissible concrete conditions					Permissible drilling method		
		Uncracked concrete		Dry concrete			Hammer drilling with carbide tipped drill bit
		Cracked concrete		Water saturated concrete			Hilti TE-CD or TE-YD Hollow Drill Bit

3.2.2

Table 38 - Hilti HAS threaded rod specifications

Setting information		Symbol	Units	Nominal rod diameter, d 3/8	1/2	5/8	3/4	7/8	1	1-1/4
Nominal bit diameter		d_o	in.	7/16	9/16	3/4	7/8	1	1-1/8	1-3/8
Effective embedment	minimum	$h_{ef,min}$	in. (mm)	2-3/8 (60)	2-3/4 (70)	3-1/8 (79)	3-1/2 (89)	3-1/2 (89)	4 (102)	5 (127)
	maximum	$h_{ef,max}$	in. (mm)	7-1/2 (191)	10 (254)	12-1/2 (318)	15 (381)	17-1/2 (445)	20 (508)	25 (635)
Diameter of fixture hole	through-set		in.	1/2	5/8	13/16[1]	15/16[1]	1-1/8[1]	1-1/4[1]	1-1/2[1]
Diameter of fixture hole	preset		in.	7/16	9/16	11/16	13/16	15/16	1-1/8	1-3/8
Installation torque		T_{inst}	ft-lb (Nm)	15 (20)	30 (40)	60 (80)	100 (136)	125 (169)	150 (203)	200 (271)
Minimum concrete thickness		h_{min}	in. (mm)	h_{ef}+1-1/4 (h_{ef}+30)		$h_{ef}+2d_o$				
Minimum edge distance		c_{min}	in. (mm)	1-3/4 (45)	1-3/4 (45)	2[2] (50)[2]	2-1/8[2] (55)[2]	2-1/4[2] (60)[2]	2-3/4[2] (70)[2]	3-1/8[2] (80)[2]
Minimum anchor spacing		s_{min}	in. (mm)	1-7/8 (48)	2-1/2 (64)	3-1/8 (79)	3-3/4 (95)	4-3/4 (111)	5 (127)	6-1/4 (159)

1 Install using (2) washers. See Figure 11.
2 Edge distance of 1-3/4-inch (44mm) is permitted provided the installation torque is reduced to 0.30 T_{inst} for 5d < s < 16-in. and to 0.5 T_{inst} for s > 16-in.

Figure 10 - Hilti HAS threaded rods

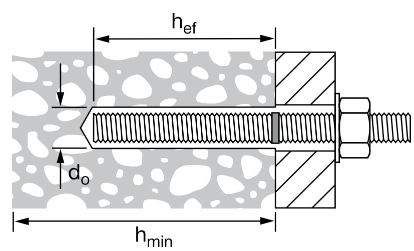


Figure 11 - Installation with (2) washers

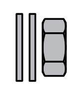


Table 39 - Hilti HIT-HY 200 adhesive design strength with concrete / bond failure for threaded rod in uncracked concrete [1,2,3,4,5,6,7,8,9]

Nominal anchor diameter in.	Effective embedment in. (mm)	Tension — ΦN_n				Shear — ΦV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
3/8	2-3/8 (60)	2,855 (12.7)	3,125 (13.9)	3,610 (16.1)	4,405 (19.6)	3,075 (13.7)	3,370 (15.0)	3,890 (17.3)	4,745 (21.1)
	3-3/8 (86)	4,835 (21.5)	5,300 (23.6)	6,015 (26.8)	6,260 (27.8)	10,415 (46.3)	11,410 (50.8)	12,950 (57.6)	13,490 (60.0)
	4-1/2 (114)	7,445 (33.1)	7,790 (34.7)	8,020 (35.7)	8,350 (37.1)	16,035 (71.3)	16,780 (74.6)	17,270 (76.8)	17,985 (80.0)
	7-1/2 (191)	12,750 (56.7)	12,985 (57.8)	13,365 (59.5)	13,915 (61.9)	27,460 (122.1)	27,965 (124.4)	28,785 (128.0)	29,975 (133.3)
1/2	2-3/4 (70)	3,555 (15.8)	3,895 (17.3)	4,500 (20.0)	5,510 (24.5)	7,660 (34.1)	8,395 (37.3)	9,690 (43.1)	11,870 (52.8)
	4-1/2 (114)	7,445 (33.1)	8,155 (36.3)	9,420 (41.9)	11,135 (49.5)	16,035 (71.3)	17,570 (78.2)	20,285 (90.2)	23,980 (106.7)
	6 (152)	11,465 (51.0)	12,560 (55.9)	14,255 (63.4)	14,845 (66.0)	24,690 (109.8)	27,045 (120.3)	30,700 (136.6)	31,970 (142.2)
	10 (254)	22,665 (100.8)	23,085 (102.7)	23,755 (105.7)	24,740 (110.0)	48,820 (217.2)	49,720 (221.2)	51,170 (227.6)	53,285 (237.0)
5/8	3-1/8 (79)	4,310 (19.2)	4,720 (21.0)	5,450 (24.2)	6,675 (29.7)	9,280 (41.3)	10,165 (45.2)	11,740 (52.2)	14,380 (64.0)
	5-5/8 (143)	10,405 (46.3)	11,400 (50.7)	13,165 (58.6)	16,120 (71.7)	22,415 (99.7)	24,550 (109.2)	28,350 (126.1)	34,720 (154.4)
	7-1/2 (191)	16,020 (71.3)	17,550 (78.1)	20,265 (90.1)	23,195 (103.2)	34,505 (153.5)	37,800 (168.1)	43,650 (194.2)	49,955 (222.2)
	12-1/2 (318)	34,470 (153.3)	36,070 (160.4)	37,120 (165.1)	38,655 (171.9)	74,245 (330.3)	77,685 (345.6)	79,955 (355.7)	83,260 (370.4)
3/4	3-1/2 (89)	5,105 (22.7)	5,595 (24.9)	6,460 (28.7)	7,910 (35.2)	11,000 (48.9)	12,050 (53.6)	13,915 (61.9)	17,040 (75.8)
	6-3/4 (171)	13,680 (60.9)	14,985 (66.7)	17,305 (77.0)	21,190 (94.3)	29,460 (131.0)	32,275 (143.6)	37,265 (165.8)	45,645 (203.0)
	9 (229)	21,060 (93.7)	23,070 (102.6)	26,640 (118.5)	32,625 (145.1)	45,360 (201.8)	49,690 (221.0)	57,375 (255.2)	70,270 (312.6)
	15 (381)	45,315 (201.6)	49,640 (220.8)	53,455 (237.8)	55,665 (247.6)	97,600 (434.1)	106,915 (475.6)	115,130 (512.1)	119,895 (533.3)
7/8	3-1/2 (89)	5,105 (22.7)	5,595 (24.9)	6,460 (28.7)	7,910 (35.2)	11,000 (48.9)	12,050 (53.6)	13,915 (61.9)	17,040 (75.8)
	7-7/8 (200)	17,235 (76.7)	18,885 (84.0)	21,805 (97.0)	26,705 (118.8)	37,125 (165.1)	40,670 (180.9)	46,960 (208.9)	57,515 (255.8)
	10-1/2 (267)	26,540 (118.1)	29,070 (129.3)	33,570 (149.3)	41,115 (182.9)	57,160 (254.3)	62,615 (278.5)	72,300 (321.6)	88,550 (393.9)
	17-1/2 (445)	57,100 (254.0)	62,550 (278.2)	72,230 (321.3)	75,770 (337.0)	122,990 (547.1)	134,730 (599.3)	155,570 (692.0)	163,190 (725.9)
1	4 (102)	6,240 (27.8)	6,835 (30.4)	7,895 (35.1)	9,665 (43.0)	13,440 (59.8)	14,725 (65.5)	17,000 (75.6)	20,820 (92.6)
	9 (229)	21,060 (93.7)	23,070 (102.6)	26,640 (118.5)	32,625 (145.1)	45,360 (201.8)	49,690 (221.0)	57,375 (255.2)	70,270 (312.6)
	12 (305)	32,425 (144.2)	35,520 (158.0)	41,015 (182.4)	50,230 (223.4)	69,835 (310.6)	76,500 (340.3)	88,335 (392.9)	108,190 (481.3)
	20 (508)	69,765 (310.3)	76,425 (340.0)	88,245 (392.5)	98,960 (440.2)	150,265 (668.4)	164,605 (732.2)	190,070 (845.5)	213,150 (948.1)
1-1/4	5 (127)	8,720 (38.8)	9,555 (42.5)	11,030 (49.1)	13,510 (60.1)	18,785 (83.6)	20,575 (91.5)	23,760 (105.7)	29,100 (129.4)
	11-1/4 (286)	29,430 (130.9)	32,240 (143.4)	37,230 (165.6)	45,595 (202.8)	63,395 (282.0)	69,445 (308.9)	80,185 (356.7)	98,205 (436.8)
	15 (381)	45,315 (201.6)	49,640 (220.8)	57,320 (255.0)	70,200 (312.3)	97,600 (434.1)	106,915 (475.6)	123,455 (549.2)	151,200 (672.6)
	25 (635)	97,500 (433.7)	106,805 (475.1)	123,330 (548.6)	151,045 (671.9)	210,000 (934.1)	230,045 (1023.3)	265,630 (1181.6)	325,330 (1447.1)

1 See section 3.1.8 for explanation on development of load values.
2 See section 3.1.8 to convert design strength (factored resistance) value to ASD value.
3 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.
4 Apply spacing, edge distance, and concrete thickness factors in tables 42 - 55 as necessary to the above values. Compare to the steel values in table 41. The lesser of the values is to be used for the design.
5 Data is for temperature range A: Max. short term temperature = 130° F (55° C), max. long term temperature = 110° F (43° C).
For temperature range B: Max. short term temperature = 176° F (80° C), max. long term temperature = 110° F (43° C) multiply above values by 0.92.
For temperature range C: Max. short term temperature = 248° F (120° C), max. long term temperature = 162° F (72° C) multiply above values by 0.78.
Short term elevated concrete temperatures are those that occur over brief intervals, e.g., as a result of diurnal cycling. Long term concrete temperatures are roughly constant over significant periods of time.
6 Tabular values are for dry and water saturated concrete conditions.
7 Tabular values are for short term loads only. For sustained loads including overhead use, see section 3.1.8.
8 Tabular values are for normal-weight concrete only. For lightweight concrete, multiply design strength (factored resistance) by λ_a as follows:
For sand-lightweight, λ_a = 0.51. For all-lightweight, λ_a = 0.45.
9 Tabular values are for static loads only. Seismic design is not permitted for uncracked concrete.

Table 41 - Steel design strength for Hilti HAS threaded rods for use with ACI 318-14 Chapter 17

Nominal anchor diameter in.	HAS-V-36 / HAS-V-36 HDG ASTM F1554 Gr. 55[4,5]			HAS-E-55 / HAS-E-55 HDG ASTM F1554 Gr. 55[4,5,6]			HAS-B-105 and HAS-B-105 HDG ASTM A193 B7 and ASTM F 1554 Gr.105[4]			HAS-R stainless steel ASTM F593 (3/8-in to 1-in)[5] ASTM A193 (1-1/8-in to 2-in)[4]		
	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)
3/8	3,370 (15.0)	1,750 (7.8)	1,050 (4.7)	4,360 (19.4)	2,270 (10.1)	1,590 (7.1)	7,270 (32.3)	3,780 (16.8)	2,645 (11.8)	5,040 (22.4)	2,790 (12.4)	1,955 (8.7)
1/2	6,175 (27.5)	3,210 (14.3)	1,925 (8.6)	7,985 (35.5)	4,150 (18.5)	2,905 (12.9)	13,305 (59.2)	6,920 (30.8)	4,845 (21.6)	9,225 (41.0)	5,110 (22.7)	3,575 (15.9)
5/8	9,835 (43.7)	5,110 (22.7)	3,065 (13.6)	12,715 (56.6)	6,610 (29.4)	4,625 (20.6)	21,190 (94.3)	11,020 (49.0)	7,715 (34.3)	14,690 (65.3)	8,135 (36.2)	5,695 (25.3)
3/4	14,550 (64.7)	7,565 (33.7)	4,540 (20.2)	18,820 (83.7)	9,785 (43.5)	6,850 (30.5)	31,360 (139.5)	16,310 (72.6)	11,415 (50.8)	18,485 (82.2)	10,235 (45.5)	7,165 (31.9)
7/8	20,085 (89.3)	10,445 (46.5)	6,265 (27.9)	25,975 (115.5)	13,505 (60.1)	9,455 (42.1)	43,285 (192.5)	22,510 (100.1)	15,755 (70.1)	25,510 (113.5)	14,125 (62.8)	9,890 (44.0)
1	26,350 (117.2)	13,700 (60.9)	8,220 (36.6)	34,075 (151.6)	17,720 (78.8)	12,405 (55.2)	56,785 (252.6)	29,530 (131.4)	20,670 (91.9)	33,465 (148.9)	18,535 (82.4)	12,975 (57.7)
1-1/4	42,160 (187.5)	21,920 (97.5)	13,150 (58.5)	54,515 (242.5)	28,345 (126.1)	19,840 (88.3)	90,855 (404.1)	47,245 (210.2)	33,070 (147.1)	41,430 (184.3)	21,545 (95.8)	12,925 (57.5)

1 Tensile = $\phi A_{se,N} f_{uta}$ as noted in ACI 318-14 17.4.1.2
2 Shear = $\phi\, 0.60 A_{se,V} f_{uta}$ as noted in ACI 318-14 17.5.1.2b.
3 Seismic Shear = $\alpha_{V,seis}\, \phi V_{sa}$: Reduction factor for seismic shear only. See ACI 318 for additional information on seismic applications.
4 HAS-V, HAS-E (3/8-in to 1-1/4-in), HAS-B, and HAS-R (Class 1; 1-1/4-in) threaded rods are considered ductile steel elements (including HDG rods).
5 HAS-R (CW1 and CW2; 3/8-in to 1-in) threaded rods are considered brittle steel elements.
6 3/8-inch dia. threaded rods are not included in the ASTM F1554 standard. Hilti 3/8-inch dia. HAS-V, HAS-E, and HAS-E-B (incl. HDG) threaded rods meet the chemical composition and mechanical property requirements of ASTM F1554.

Foundations

RECTANGULAR SPREAD FOOTING ANALYSIS

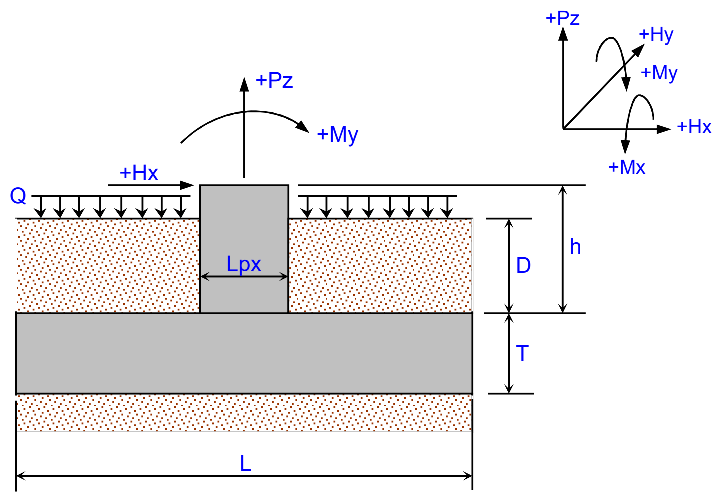
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Porch Footer J&K - 7.7		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data:

Footing Data:

Footing Length, L =	20.000	ft.
Footing Width, B =	10.000	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

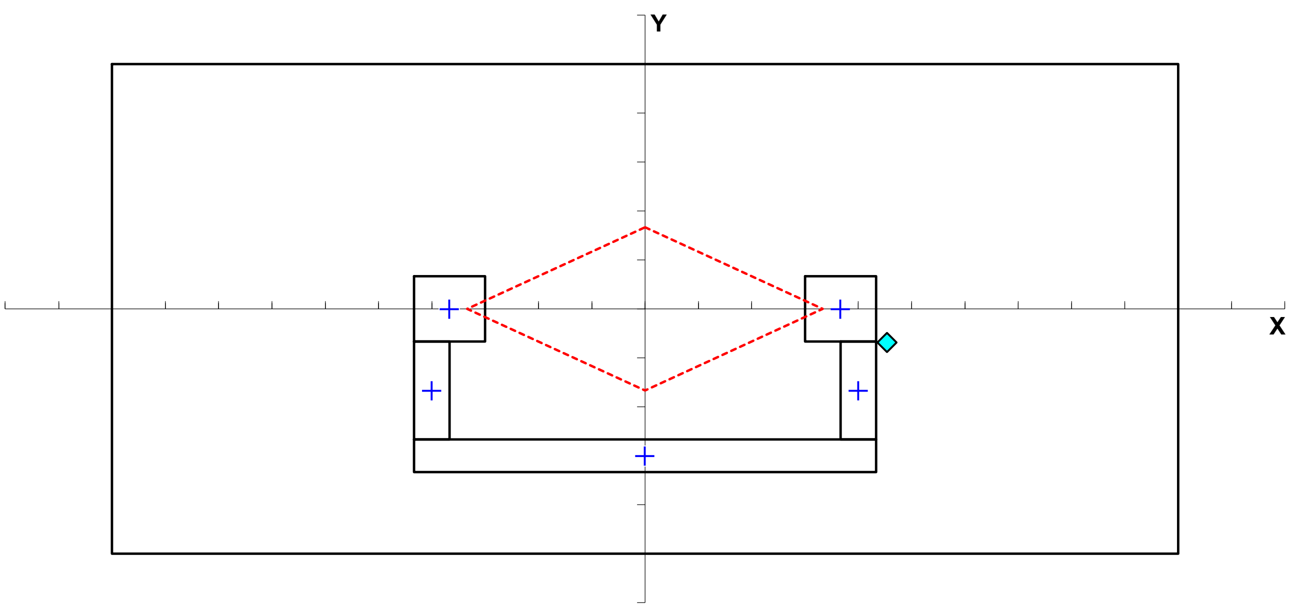


Nomenclature

Pier/Loading Data:

Number of Piers = 5

	Pier #1	Pier #2	Pier #3	Pier #4	Pier #5			
Xp (ft.) =	-3.667	3.667	0.000	-4.000	4.000			
Yp (ft.) =	0.000	0.000	-3.000	-1.667	-1.667			
Lpx (ft.) =	1.333	1.333	8.667	0.667	0.667			
Lpy (ft.) =	1.333	1.333	0.667	2.000	2.000			
h (ft.) =	0.000	0.000	0.000	0.000	0.000			
Pz (k) =	22.691	-42.85	-28.34	-1.18	-6.62			
Hx (k) =	15.604	16.92	0.00	0.00	0.00			
Hy (k) =	0.000	0.00	0.00	0.00	0.00			
Mx (ft-k) =	0.000	0.00	0.00	0.00	0.00			
My (ft-k) =	157.465	167.11	0.00	0.00	0.00			



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-143.63	kips
ex =	4.54	ft. (> L/6)
ey =	-0.68	ft. (<= B/6)

Overturning Check:

ΣMrx =	733.58	ft-kips
ΣMox =	113.46	ft-kips
FS(ot)x =	6.466	>= 1.5
ΣMry =	1484.29	ft-kips
ΣMoy =	699.75	ft-kips
FS(ot)y =	2.121	>= 1.5

Sliding Check:

Pass(x) =	15.40	kips
Frict(x) =	57.45	kips
FS(slid)x =	2.240	>= 1.5
Pass(y) =	30.80	kips
Frict(y) =	57.45	kips
FS(slid)y =	N.A.	

Uplift Check:

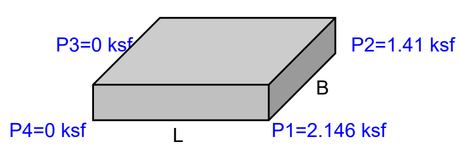
ΣPz(down) =	-166.32	kips
ΣPz(uplift) =	22.69	kips
FS(uplift) =	7.330	>= 1.5

Bearing Length and % Bearing Area:

Dist. x =	19.228	ft.
Dist. y =	29.165	ft.
Brg. Lx1 =	12.635	ft.
Brg. Lx2 =	19.228	ft.
%Brg. Area =	79.66	%
Biaxial Case =	Case 3	6*ex/L + 6*ey/B = 1.77

Gross Soil Bearing Corner Pressures:

P_1 =	2.146	ksf
P_2 =	1.410	ksf
P_3 =	0.000	ksf
P_4 =	0.000	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

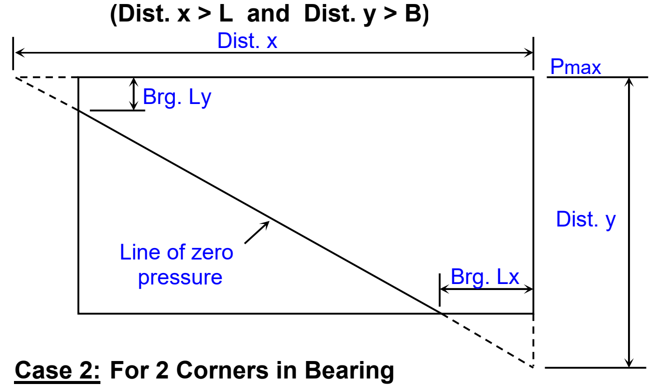
$P_{max}(net) = P_{max}(gross) - (D+T) \cdot \gamma_s$

$P_{max}(net)$ =	1.779	ksf

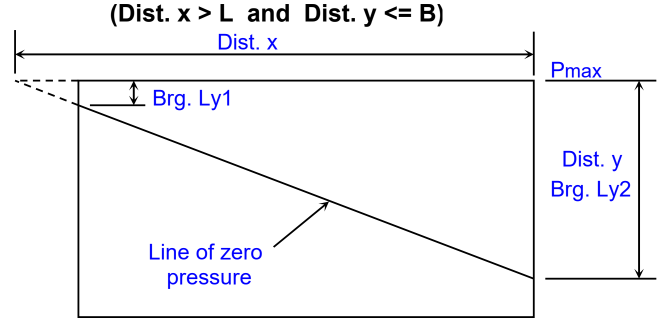
Comments:

Nomenclature for Biaxial Eccentricity:

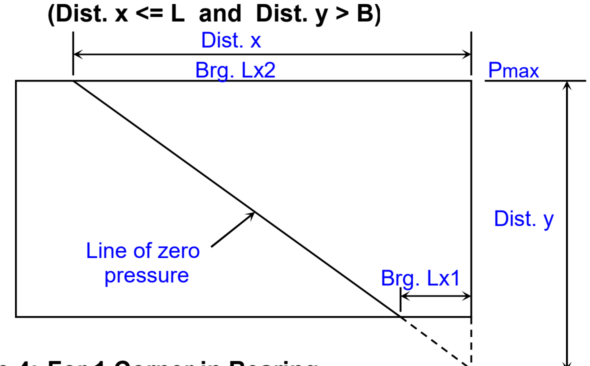
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



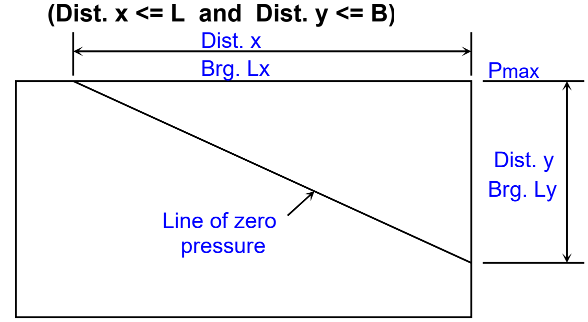
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

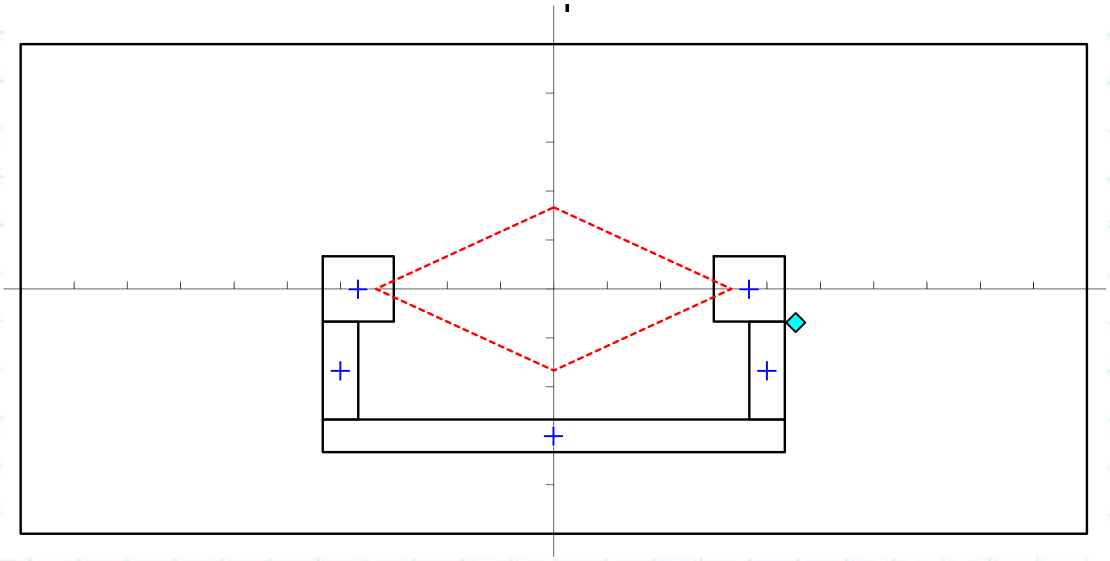


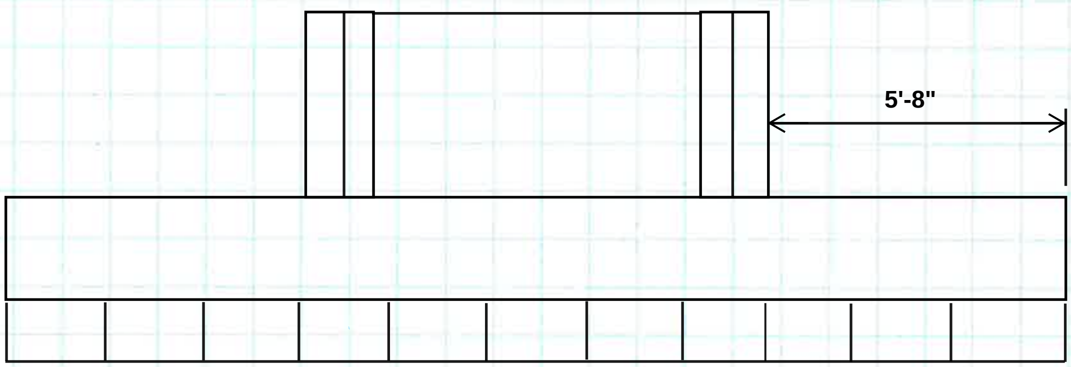
SUBJECT Check Existing Support Girders for Loads in CC4

BY EJM DATE PROJ. NO. C131942.38

CHKD. BY DATE SHEET NO. OF

gai consultants





Footing t=24" $w_F = 2(150) = 300$ psf

Soil t=16" $w_S = 1.333(110) = 147$ psf

Net soil pressure = 2100-300-147 = 1653 psf

$M_{FTG} = 1.653(5.667^2/2)(10) = 265.4$ k-ft

$M_U = 265.4(1.5) = 398.1$ k-ft

u**Purpose:** To find the required area of steel for the concrete structure given the ultimate design moment (Mx).

To Find A_s Given M_u

Input:

$f'_c \quad 4\frac{kip}{in^2}$

$F_y \quad 60\frac{kip}{in^2}$

$\phi_u \quad 0.9$

$M_u \quad 398.1\,kip\ ft$

$b \quad 120in$

$d \quad 20.5in$

Develope a Root Equation for Area of Steel:

$M_u = \phi M_n$ Therefore $M_n = M_u/\phi$

$M_n = A_sF_y(d\text{-}a/2)$ $\quad a = A_sF_y/0.85f'_cb$

$M_u/\phi = A_sF_y\{d\text{-}(A_sF_y/2(0.85)f'_cb)\}$

$M_u/\phi = A_sF_yd - (A_sF_y)^2 / 2(0.85)f'_cb$ Let $x = A_sF_y$

$M_u/\phi = dx - x^2 / 2(0.85)f'_cb$

$x^2(1 / 2(0.85)f'_cb) - xd + M_u/\phi$

Solve for x Then $A_s = x/F_y$

Constants for the Root Equation:

$A_u \quad 2 \quad 0.85 \quad f'_c \quad b \quad 816 \quad \frac{kip}{in}$

$B_u \quad d \quad 20.5 \quad in$

$C_u \quad \frac{M_u}{\phi_u} \quad 5308 \quad in \quad kip$

Solving the Root Equation:

$$f(x) := \frac{x^2}{A_u} - B_u \cdot x + C_u$$

$x := 0.01 kip$

$r0 := root(f(x), x) = 263.064 \; kip$

Required Area of Steel:

$$A_{s1} := \frac{r0}{F_y} = 4.384 \; in^2$$

Steel Used 11 ~ #6 bars $A_s = 4.84\ in^2 > A_{s1}$

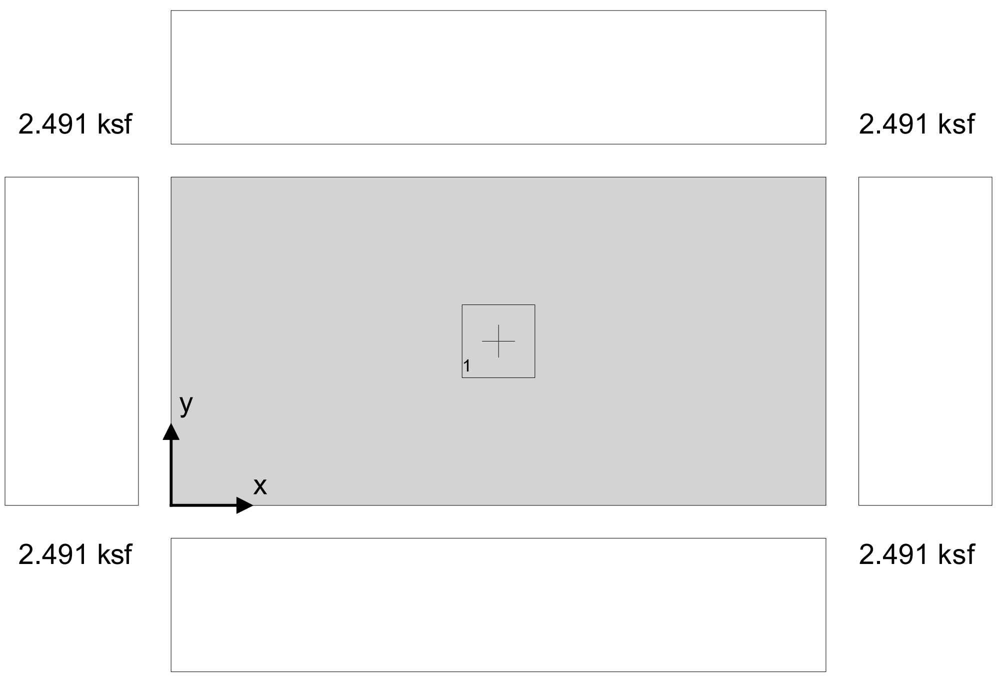
$A_{smin} := 0.0018 \cdot b \cdot d = 4.428 \; in^2$

Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **6** ft
Width of foundation	L_y = **3** ft
Foundation area	$A = L_x \times L_y$ = **18** ft^2
Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **8.00** in
Width of column	l_{y1} = **8.00** in
position in x-axis	x_1 = **36.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.546** ksf
Density of soil	γ_{soil} = **110.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Passive pressure coefficient (Rankine)	$K_P = (1 + \sin(\phi_b)) / (1 - \sin(\phi_b))$ = **3**

Self weight $F_{swt} = h \times \gamma_{conc} = \mathbf{290}$ psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil} = \mathbf{146.7}$ psf

Column no.1 loads

Dead load in z $F_{Dz1} = \mathbf{31.5}$ kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.858)

1.0D + 1.0L (0.858)

1.0D + 1.0Lr (0.858)

1.0D + 1.0S (0.858)

1.0D + 1.0R (0.858)

1.0D + 0.75L + 0.75Lr (0.858)

1.0D + 0.75L + 0.75S (0.858)

1.0D + 0.75L + 0.75R (0.858)

1.0D + 0.6W (0.858)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.979)

1.0D + 0.75L + 0.75Lr + 0.45W (0.858)

1.0D + 0.75L + 0.75S + 0.45W (0.858)

1.0D + 0.75L + 0.75R + 0.45W (0.858)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.944)

0.6D + 0.6W (0.515)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.395)

Combination 10 results: $(1.0 + 0.14 \times S_{DS})$D + 0.7E

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} = \mathbf{44.8}$ kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) = \mathbf{134.5}$ kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) = \mathbf{67.3}$ kip_ft

Uplift verification

Vertical force $F_{dz} = \mathbf{44.846}$ kips

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.491}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.491}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.491}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.491}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{2.491}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.491** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2.546** ksf

q_{max} / q_{allow} = **0.979**

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FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.080)

1.2D + 1.6L + 0.5Lr (0.068)

1.2D + 1.6L + 0.5S (0.068)

1.2D + 1.6L + 0.5R (0.068)

1.2D + 1.0L + 1.6Lr (0.068)

1.2D + 1.0L + 1.6S (0.068)

1.2D + 1.0L + 1.6R (0.068)

1.2D + 1.6Lr + 0.5W (0.068)

1.2D + 1.6S + 0.5W (0.068)

1.2D + 1.6R + 0.5W (0.068)

1.2D + 1.0L + 0.5Lr + 1.0W (0.068)

1.2D + 1.0L + 0.5S + 1.0W (0.068)

1.2D + 1.0L + 0.5R + 1.0W (0.068)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.080)

0.9D + 1.0W (0.051)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.040)

Combination 1 results: 1.4D

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1}$ = **55.1** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1)$ = **165.2** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1)$ = **82.6** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.06** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.06** ksf

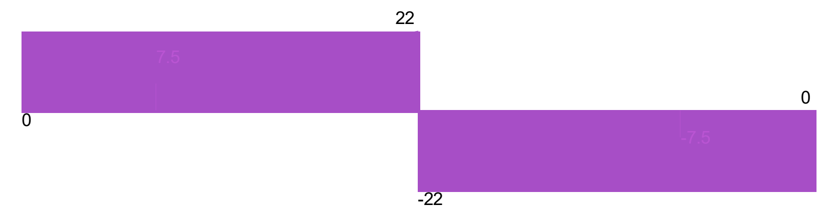
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.06** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.06** ksf

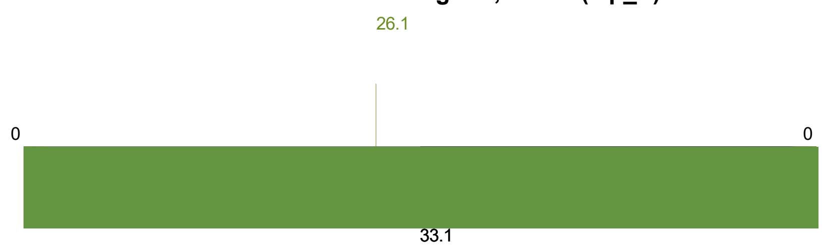
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.06** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.06** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max}$ = **26.122** kip_ft

Tension reinforcement provided — 6 No.5 bottom bars (5.8 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov}$ = **1.86** in^2

Minimum area of reinforcement (10.5.4) — $A_{s.min} = 0.0018 \times L_y \times h$ = **1.555** in^2

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Maximum spacing of reinforcement (10.5.4) — $s_{max} = \min(3 \times h, 18 \text{ in})$ = **18** in

, ,   ,   ,   ,   ,   ,   ,

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} / 2$ = **20.687** in

Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.912** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **1.073** in

Strain in tensile reinforcement (10.3.5) — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05486**

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Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **188.154** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **169.339** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.154**

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One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **7.462** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2)$ = **20.687** in

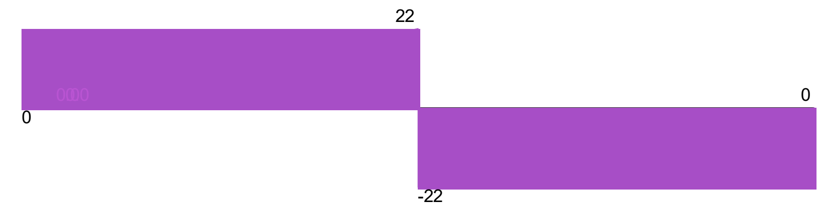
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 11-3) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v$ = **94.204** kips

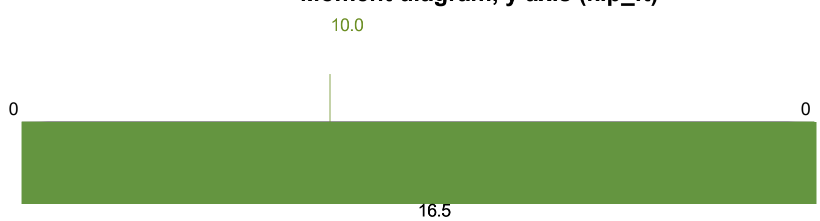
Design shear capacity $\phi V_n = \phi_v \times V_n$ = **70.653** kips

$V_{u.x} / \phi V_n$ = **0.106**

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **10.003** kip_ft

Tension reinforcement provided 6 No.7 bottom bars (13.0 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **3.6** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_x \times h$ = **3.11** in^2

Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18 \text{ in})$ = **18** in

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **19.937** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.882** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.038** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05462**

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **350.934** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **315.84** kip_ft

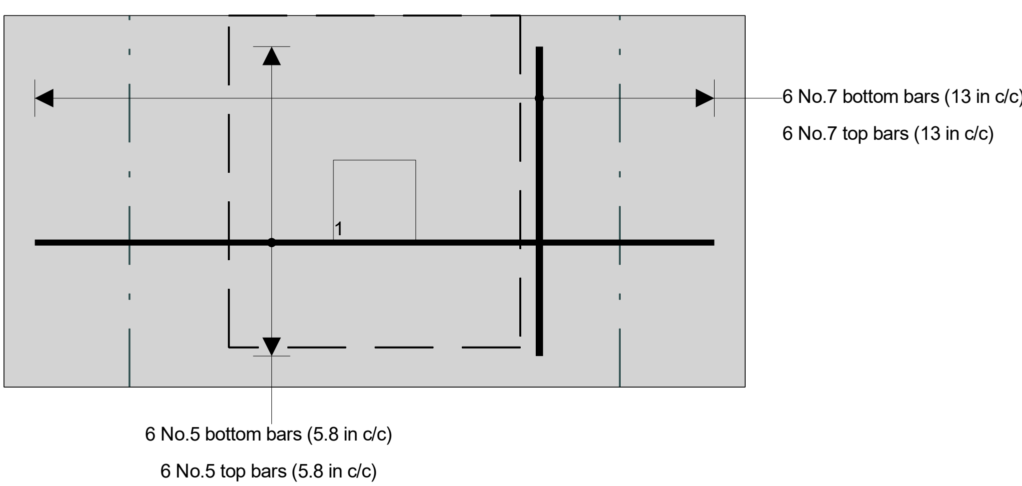
$M_{u.y.max} / \phi M_n$ = **0.032**

Footing geometry factor (15.4.4.2)	$\beta_f = L_x / L_y =$ **2.000**
Area of reinf. req. for uniform distribution (CRSI)	$A_{sreq} = (M_{u.y.max} / (\phi_f \times f_y \times (d - a / 2))) \times 2 \times \beta_f / (\beta_f + 1) =$ **0.152** in^2

One-way shear design, y direction

Two-way shear design at column 1

Depth to reinforcement	$d_{v2} =$ **20.313** in
Shear perimeter length (11.11.1.2)	$l_{xp} =$ **28.312** in
Shear perimeter width (11.11.1.2)	$l_{yp} =$ **32.156** in
Shear perimeter (11.11.1.2)	$b_o = l_{x,perim} + 2 \times l_{y,perim} =$ **92.625** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} =$ **910.424** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **846.424** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **3.060** ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **28.500** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\ \text{psi}) =$ **15.148** psi
Column geometry factor (11.11.2.1)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (11.11.2.1)	$\alpha_s =$**30**
Concrete shear strength (11.11.2.1)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ \text{psi})} =$ **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ \text{psi})} =$ **542.580** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ \text{psi})} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 11-2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (Eq. 11-1)	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.080**



RECTANGULAR SPREAD FOOTING ANALYSIS

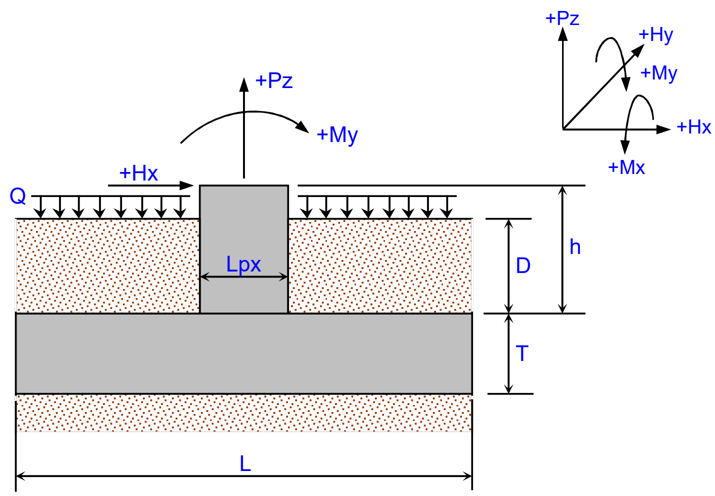
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Porch Footer J.6-4.7		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data:

Footing Data:

Footing Length, L =	6.000	ft.
Footing Width, B =	3.000	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

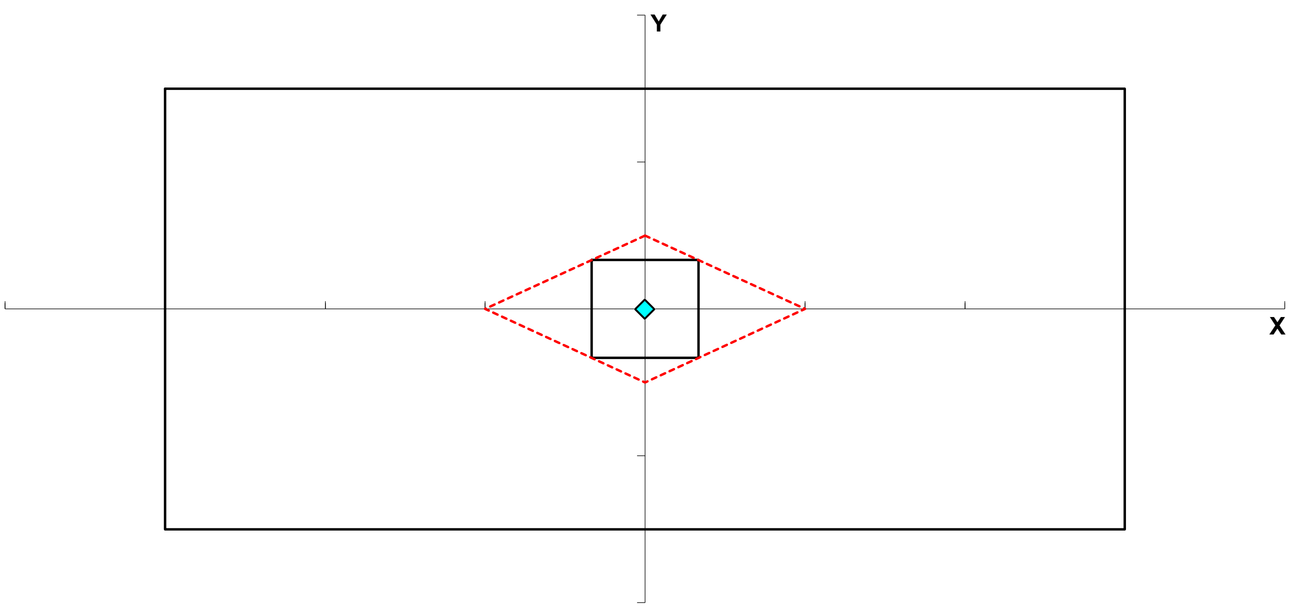


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	0.667							
Lpy (ft.) =	0.667							
h (ft.) =	0.000							
Pz (k) =	-31.479							
Hx (k) =	0.000							
Hy (k) =	0.000							
Mx (ft-k) =	0.000							
My (ft-k) =	0.000							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-39.34	kips
ex =	0.00	
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	N.A.	ft-kips
ΣMoy =	N.A.	ft-kips
FS(ot)y =	N.A.	

Sliding Check:

Pass(x) =	4.62	kips
Frict(x) =	15.74	kips
FS(slid)x =	N.A.	
Pass(y) =	9.24	kips
Frict(y) =	15.74	kips
FS(slid)y =	N.A.	

Uplift Check:

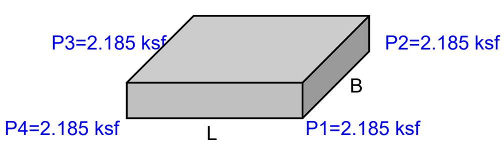
ΣPz(down) =	-39.34	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	6.000	ft.
Brg. Ly =	3.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	2.185	ksf
P_2 =	2.185	ksf
P_3 =	2.185	ksf
P_4 =	2.185	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

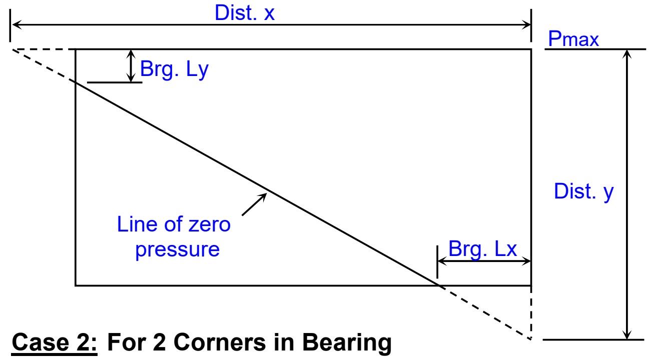
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

P_{max}(net) =	1.819	ksf

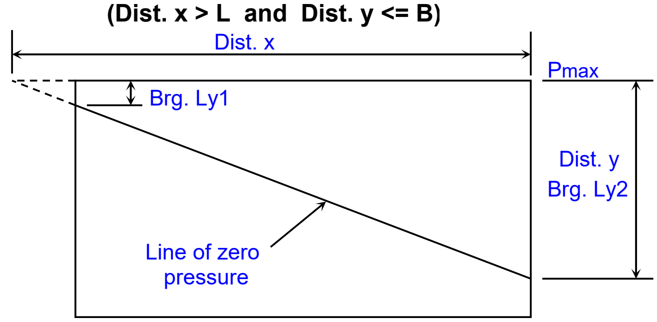
Comments:

Nomenclature for Biaxial Eccentricity:

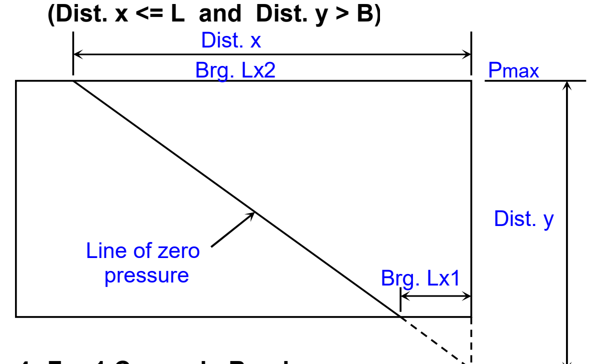
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



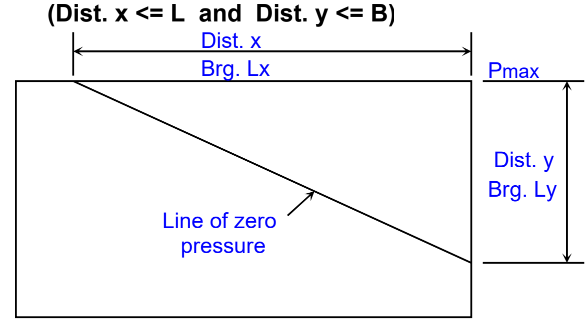
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

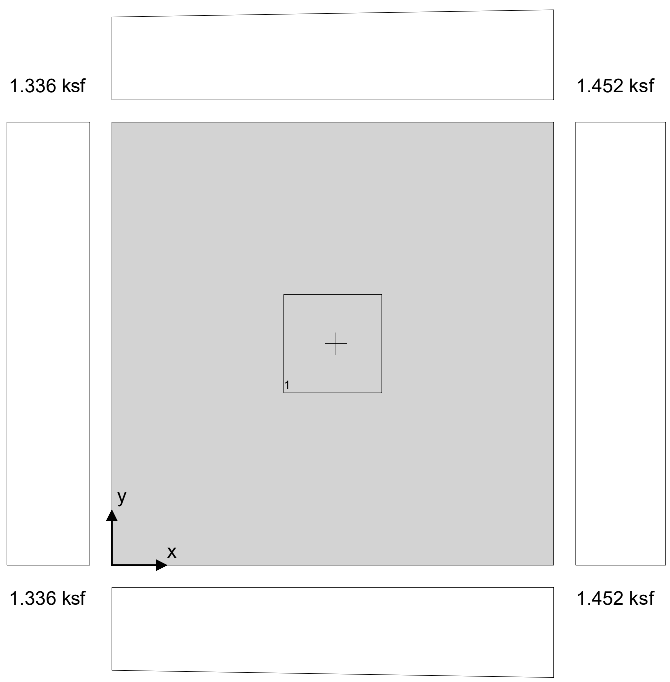


Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **6** ft
Width of foundation	L_y = **6** ft
Foundation area	$A = L_x \times L_y$ = **36** ft^2
Depth of foundation	h = **36** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **16.00** in
Width of column	l_{y1} = **16.00** in
position in x-axis	x_1 = **36.00** in
position in y-axis	y_1 = **36.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.577** ksf
Density of soil	γ_{soil} = **110.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Passive pressure coefficient (Rankine)	$K_P = (1 + \sin(\phi_b)) / (1 - \sin(\phi_b))$ = **3**

Self weight $F_{swt} = h \times \gamma_{conc} =$ **435** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil} =$ **146.7** psf

Column no.1 loads Note, weight of CMU wall added to the column dead load

Dead load in z $F_{Dz1} =$ **23.1** kips

Live roof load in z $F_{Lrz1} =$ **3.7** kips

Wind load in z $F_{Wz1} =$ **1.3** kips

Dead load in x $F_{Dx1} =$ **0.6** kips

Live load in x $F_{Lx1} =$ **0.3** kips

Wind load in x $F_{Wx1} =$ **0.1** kips

Wind load moment in x $M_{Wx1} =$ **2.6** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.494)

1.0D + 1.0L (0.503)

1.0D + 1.0Lr (0.534)

1.0D + 1.0S (0.494)

1.0D + 1.0R (0.494)

1.0D + 0.75L + 0.75Lr (0.531)

1.0D + 0.75L + 0.75S (0.501)

1.0D + 0.75L + 0.75R (0.501)

1.0D + 0.6W (0.522)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.563)

1.0D + 0.75L + 0.75Lr + 0.45W (0.552)

1.0D + 0.75L + 0.75S + 0.45W (0.522)

1.0D + 0.75L + 0.75R + 0.45W (0.522)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.551)

0.6D + 0.6W (0.324)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.227)

Combination 10 results: $(1.0 + 0.14 \times S_{DS})$D + 0.7E

Forces on foundation

Force in x-axis $F_{dx} = \gamma_D \times F_{Dx1} =$ **0.7** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} =$ **50.2** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + F_{Dx1} \times h) =$ **152.6** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) =$ **150.5** kip_ft

Uplift verification

Vertical force $F_{dz} =$ **50.178** kips

00 0 0 0 0 0

Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_D \times (F_{Dx1} \times h) =$ **2.08** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) =$ **-150.53** kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL})$ = **72.276**

00 0 0 0 0 0 0 0 0

Stability against sliding

Resistance due to base friction — $F_{RFriction} = max(F_{dz}, 0\ kN) \times tan(\delta_{bb})$ = **28.97** kips

Stability against sliding in x direction

Resistance from passive soil pressure — $F_{RxPass} = 0.5 \times K_{PE} \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil}$ = **0** kips

Total sliding resistance — $F_{Rx} = F_{RFriction} + F_{RxPass}$ = **28.97** kips

Factor of safety — $abs(F_{Rx} / F_{dx})$ = **41.73**

00 0 0 0 0 0 0 0 0

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0.498** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.336** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.336** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.452** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.452** ksf

Minimum base pressure — $q_{min} = min(q_1,q_2,q_3,q_4)$ = **1.336** ksf

Maximum base pressure — $q_{max} = max(q_1,q_2,q_3,q_4)$ = **1.452** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **2.577** ksf

q_{max} / q_{allow} = **0.563**

00 0 0 0 0 0 0

FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete — f'_c = **4000** psi

Yield strength of reinforcement — f_y = **60000** psi

Cover to reinforcement — c_{nom} = **3** in

Concrete type — Normal weight

Concrete modification factor — λ = **1.00**

Column type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.019)

1.2D + 1.6L + 0.5Lr (0.018)

1.2D + 1.6L + 0.5S (0.017)

1.2D + 1.6L + 0.5R (0.017)

1.2D + 1.0L + 1.6Lr (0.020)

1.2D + 1.0L + 1.6S (0.017)

1.2D + 1.0L + 1.6R (0.017)

1.2D + 1.6Lr + 0.5W (0.020)

1.2D + 1.6S + 0.5W (0.017)

1.2D + 1.6R + 0.5W (0.017)

1.2D + 1.0L + 0.5Lr + 1.0W (0.018)

1.2D + 1.0L + 0.5S + 1.0W (0.017)

1.2D + 1.0L + 0.5R + 1.0W (0.017)

(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.019)

0.9D + 1.0W (0.013)

(0.9 - 0.2 × S_{DS})D + 1.0E (0.010)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in x-axis $F_{ux} = \gamma_D \times F_{Dx1} + \gamma_W \times F_{Wx1}$ = **0.8** kips

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **59.4** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + F_{Dx1} \times h) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1} + F_{Wx1} \times h)$ = **181.8** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **178.2** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0.741** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.548** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.548** ksf

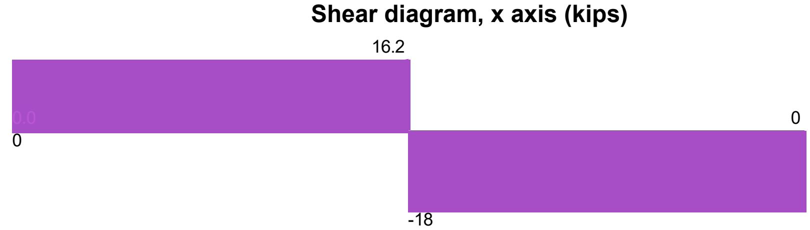
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.752** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.752** ksf

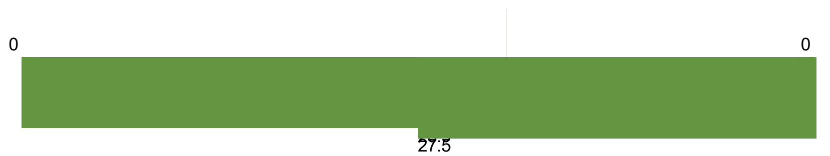
Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **1.548** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **1.752** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **16.777** kip_ft

Tension reinforcement provided 12 No.6 bottom bars (5.9 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **5.28** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_y \times h$ = **4.666** in^2

00 0 0 0 0 0

Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in

00 0 0 0 0 0 0

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2$ = **32.625** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **1.294** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.522** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06129**

00 0 0 0 0 0

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **844.218** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **759.796** kip_ft

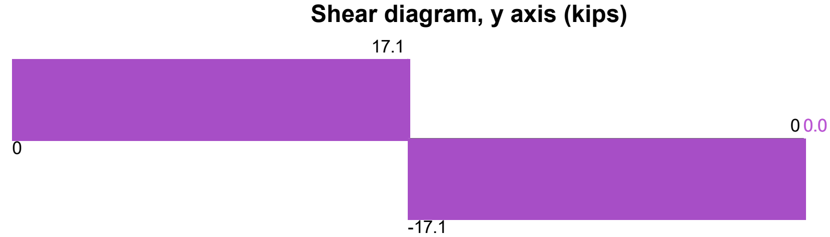
$M_{u.x.max} / \phi M_n$ = **0.022**

00 0 0 0 0 0 0

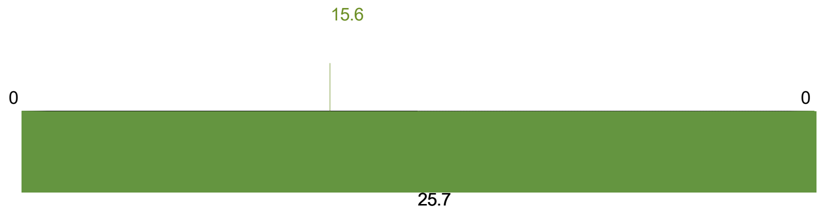
One-way shear design, x direction

0 0 0 0 0 0 0 0 0 0 0 0 0

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **15.553** kip_ft

Tension reinforcement provided — 12 No.6 bottom bars (5.9 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **5.28** in^2

Minimum area of reinforcement (10.5.4) — $A_{s.min} = 0.0018 \times L_x \times h$ = **4.666** in^2

00 0 0 0 0 0

Maximum spacing of reinforcement (10.5.4) — $s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in

00 0 0 0 0 0 0

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **31.875** in

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **1.294** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **1.522** in

Strain in tensile reinforcement (10.3.5) — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05981**

00 0 0 0 0 0

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **824.418** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **741.976** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.021**

00 0 0 0 0 0 0

One-way shear design, y direction

0 0 0 0 0 0 0 0 0 0 0 0 0

Two-way shear design at column 1

Depth to reinforcement — d_{v2} = **32.25** in

Shear perimeter length (11.11.1.2) — l_{xp} = **48.250** in

Shear perimeter width (11.11.1.2) — l_{yp} = **48.250** in

Shear perimeter (11.11.1.2) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **193.000** in

Shear area — $A_p = l_{x,perim} \times l_{y,perim}$ = **2328.063** in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **2072.063** in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg}$ = **1.650** ksf

Ultimate shear load — $F_{up} = \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **18.562** kips

Ultimate shear stress from vertical load — $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **2.982** psi

Column geometry factor (11.11.2.1) — $\beta = l_{y1} / l_{x1}$ = **1.00**

Column location factor (11.11.2.1) — α_s =**40**

Concrete shear strength (11.11.2.1) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **549.220** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi

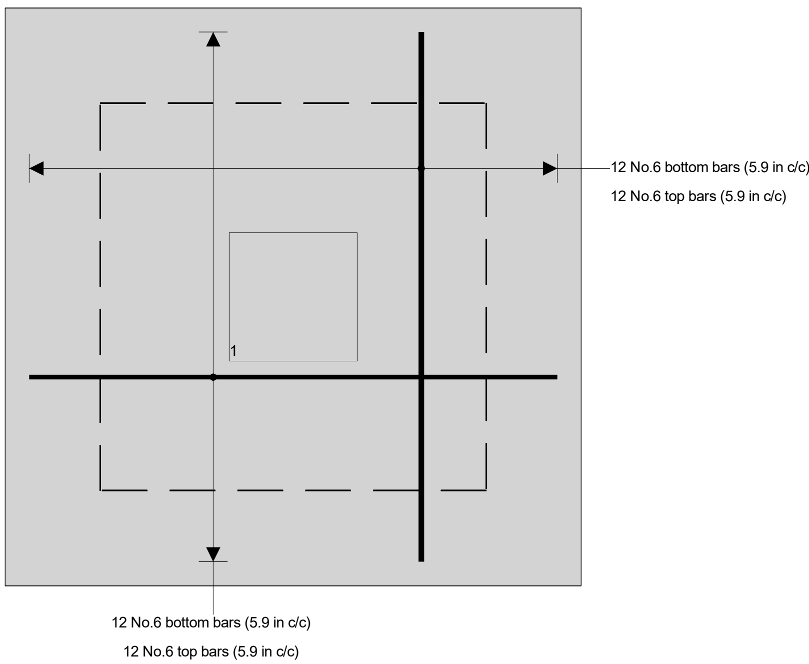
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 11-2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (Eq. 11-1) $\phi v_n = \phi_v \times v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.016**

00 0 0 0 0 0 0 0 0



RECTANGULAR SPREAD FOOTING ANALYSIS

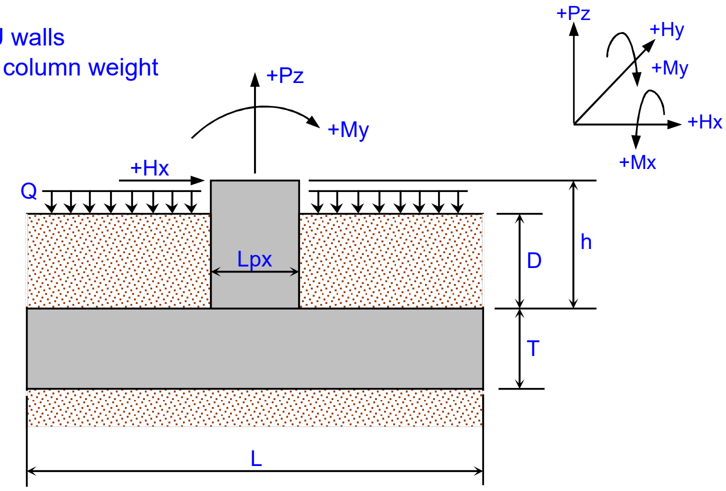
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Pavilion Footer L-3		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data: Note: 2nd & 3rd piers represent CMU walls
Pz includes ridge beam reaction and column weight

Footing Data:

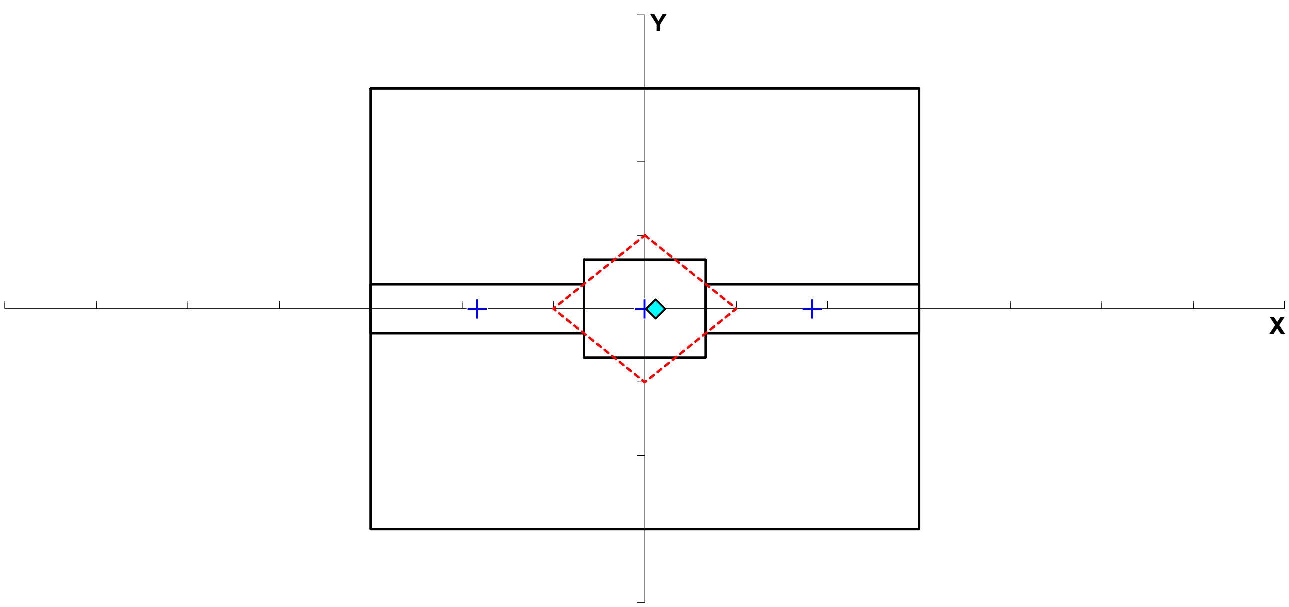
Footing Length, L =	6.000	ft.
Footing Width, B =	6.000	ft.
Footing Thickness, T =	3.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf



Nomenclature

Pier/Loading Data:
Number of Piers = 3

	Pier #1	Pier #2	Pier #3					
Xp (ft.) =	0.000	1.833	-1.833					
Yp (ft.) =	0.000	0.000	0.000					
Lpx (ft.) =	1.333	2.333	2.333					
Lpy (ft.) =	1.333	0.667	0.667					
h (ft.) =	0.000	0.000	0.000					
Pz (k) =	-23.357	-1.72	-1.72					
Hx (k) =	1.000	0.00	0.00					
Hy (k) =	0.000	0.00	0.00					
Mx (ft-k) =	0.000	0.00	0.00					
My (ft-k) =	2.644	0.00	0.00					



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-47.73	kips
ex =	0.12	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	143.20	ft-kips
ΣMoy =	5.64	ft-kips
FS(ot)y =	25.371	>= 1.5

Sliding Check:

Pass(x) =	16.83	kips
Frict(x) =	19.09	kips
FS(slid)x =	35.921	>= 1.5
Pass(y) =	16.83	kips
Frict(y) =	19.09	kips
FS(slid)y =	N.A.	

Uplift Check:

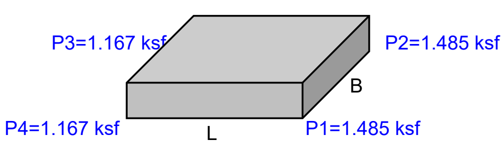
ΣPz(down) =	-47.73	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	6.000	ft.
Brg. Ly =	6.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	1.485	ksf
P_2 =	1.485	ksf
P_3 =	1.167	ksf
P_4 =	1.167	ksf



CORNER PRESSURES

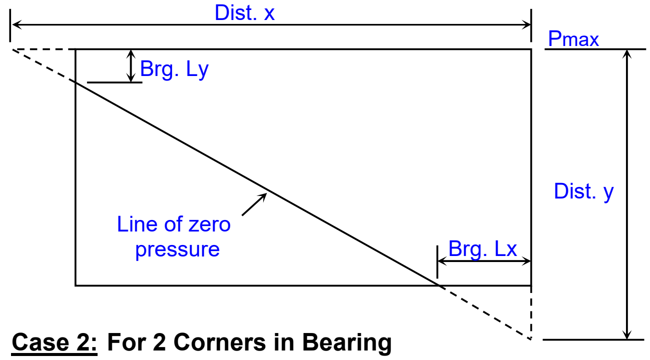
Maximum Net Soil Pressure:

$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

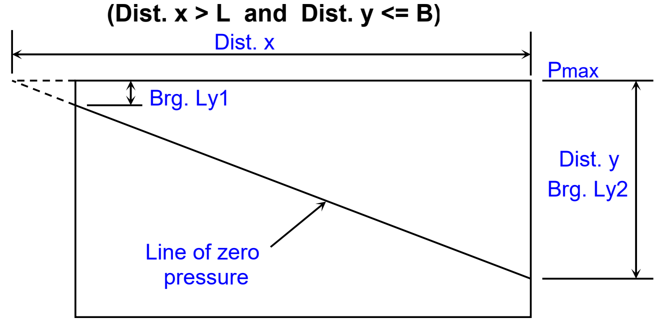
$P_{max}(net)$ =	1.008	ksf

Nomenclature for Biaxial Eccentricity:

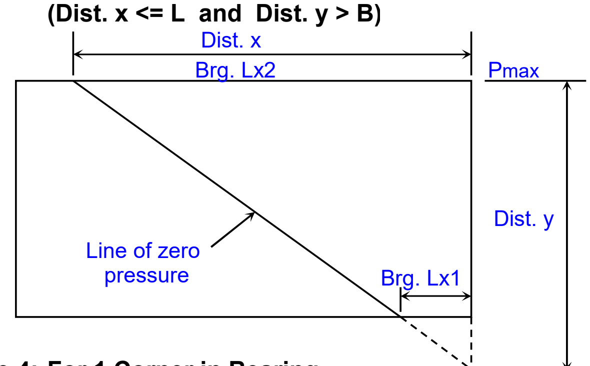
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



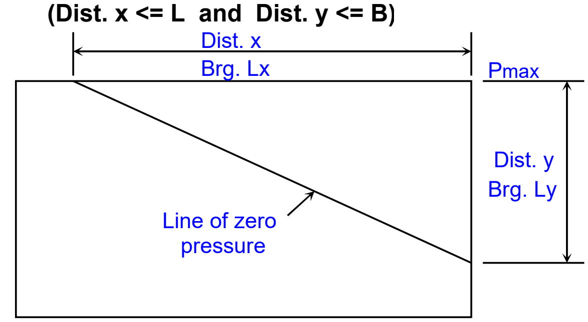
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



Comments:

Interior column, minimum lateral load of 5 psf on surface, fixed at base

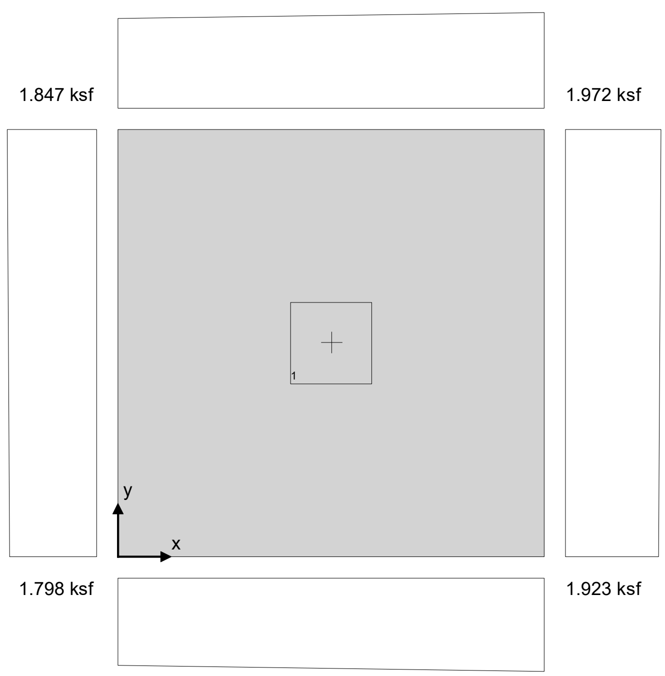
$Mx= 5(1.333)(28.167^2)/2= 2644$ lb-ft

Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **3.5** ft
Width of foundation	L_y = **3.5** ft
Foundation area	$A = L_x \times L_y$ = **12.25** ft^2
Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **8.00** in
Width of column	l_{y1} = **8.00** in
position in x-axis	x_1 = **21.00** in
position in y-axis	y_1 = **21.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.247** ksf
Density of soil	γ_{soil} = **110.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Passive pressure coefficient (Rankine)	$K_P = (1 + \sin(\phi_b)) / (1 - \sin(\phi_b))$ = **3**

Self weight	$F_{swt} = h \times \gamma_{conc} = \mathbf{290}$ psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil} = \mathbf{146.7}$ psf

Column no.1 loads

Dead load in z	$F_{Dz1} = \mathbf{13.6}$ kips
Live roof load in z	$F_{Lrz1} = \mathbf{4.6}$ kips
Wind load in z	$F_{Wz1} = \mathbf{1.7}$ kips
Wind load in x	$F_{Wx1} = \mathbf{0.3}$ kips
Dead load moment in x	$M_{Dx1} = \mathbf{0.1}$ kip_ft
Live load moment in x	$M_{Lx1} = \mathbf{0.1}$ kip_ft
Dead load moment in y	$M_{Dy1} = \mathbf{0.1}$ kip_ft
Live load moment in y	$M_{Ly1} = \mathbf{0.1}$ kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.699)
1.0D + 1.0L (0.712)
1.0D + 1.0Lr (0.865)
1.0D + 1.0S (0.699)
1.0D + 1.0R (0.699)
1.0D + 0.75L + 0.75Lr (0.833)
1.0D + 0.75L + 0.75S (0.709)
1.0D + 0.75L + 0.75R (0.709)
1.0D + 0.6W (0.759)
$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.797)
1.0D + 0.75L + 0.75Lr + 0.45W (0.878)
1.0D + 0.75L + 0.75S + 0.45W (0.753)
1.0D + 0.75L + 0.75R + 0.45W (0.753)
$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.779)
0.6D + 0.6W (0.479)
$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.322)

Combination 11 results: 1.0D + 0.75L + 0.75Lr + 0.45W

Forces on foundation

Force in x-axis	$F_{dx} = \gamma_W \times F_{Wx1} = \mathbf{0.1}$ kips
Force in z-axis	$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} = \mathbf{23.1}$ kips

Moments on foundation

Moment in x-axis, about x is 0	$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1+M_{Dx1}) + \gamma_L \times (M_{Lx1}) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1+F_{Wx1} \times h) = \mathbf{40.9}$ kip_ft
Moment in y-axis, about y is 0	$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1+M_{Dy1}) + \gamma_L \times (M_{Ly1}) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1) = \mathbf{40.6}$ kip_ft

Uplift verification

Vertical force	$F_{dz} = \mathbf{23.093}$ kips

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Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_D \times (M_{Dx1}) + \gamma_L \times (M_{Lx1}) + \gamma_W \times (F_{Wx1} \times h) =$ **0.44** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_{Lr} \times (F_{Lrz1} \times (x_1 - L_x)) + \gamma_W \times (F_{Wz1} \times (x_1 - L_x)) =$ **-40.41** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL}) =$ **90.814**

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Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_D \times (M_{Dy1}) + \gamma_L \times (M_{Ly1}) =$ **0.18** kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (F_{Dz1} \times (y_1 - L_y)) + \gamma_{Lr} \times (F_{Lrz1} \times (y_1 - L_y)) + \gamma_W \times (F_{Wz1} \times (y_1 - L_y)) =$ **-40.41** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL}) =$ **230.927**

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Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 \text{ kN}) \times tan(\delta_{bb}) =$ **13.333** kips

Stability against sliding in x direction

Resistance from passive soil pressure $F_{RxPass} = 0.5 \times K_P \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil} =$ **5.39** kips

Total sliding resistance $F_{Rx} = F_{RFriction} + F_{RxPass} =$ **18.723** kips

Factor of safety $abs(F_{Rx} / F_{dx}) =$ **138.69**

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0.231** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0.091** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.798** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.847** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.923** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.972** ksf

Minimum base pressure $q_{min} = min(q_1,q_2,q_3,q_4) =$ **1.798** ksf

Maximum base pressure $q_{max} = max(q_1,q_2,q_3,q_4) =$ **1.972** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} =$ **2.247** ksf

$q_{max} / q_{allow} =$ **0.878**

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FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete $f'_c =$ **4000** psi

Yield strength of reinforcement $f_y =$ **60000** psi

Cover to reinforcement $c_{nom} =$ **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.032)

1.2D + 1.6L + 0.5Lr (0.032)

1.2D + 1.6L + 0.5S (0.028)

1.2D + 1.6L + 0.5R (0.028)

1.2D + 1.0L + 1.6Lr (0.040)

1.2D + 1.0L + 1.6S (0.028)

1.2D + 1.0L + 1.6R (0.028)

1.2D + 1.6Lr + 0.5W (0.041)

1.2D + 1.6S + 0.5W (0.029)

1.2D + 1.6R + 0.5W (0.029)

1.2D + 1.0L + 0.5Lr + 1.0W (0.035)

1.2D + 1.0L + 0.5S + 1.0W (0.031)

1.2D + 1.0L + 0.5R + 1.0W (0.031)

$(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.2S + 1.0E$ (0.033)

0.9D + 1.0W (0.024)

$(0.9 - 0.2 \times S_{DS})D + 1.0E$ (0.017)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in x-axis $F_{ux} = \gamma_W \times F_{Wx1}$ = **0.2** kips

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **30.8** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1}) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + F_{Wx1} \times h)$ = **54.4** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1 + M_{Dy1}) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **54.1** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0.163** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.047** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.441** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.475** ksf

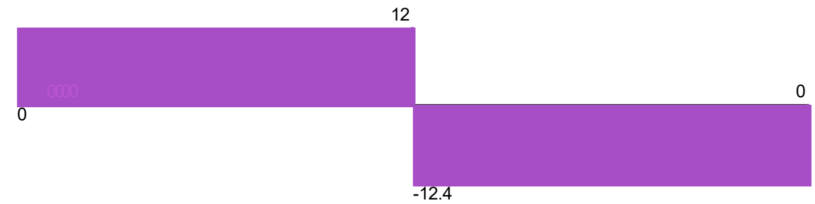
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.559** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.592** ksf

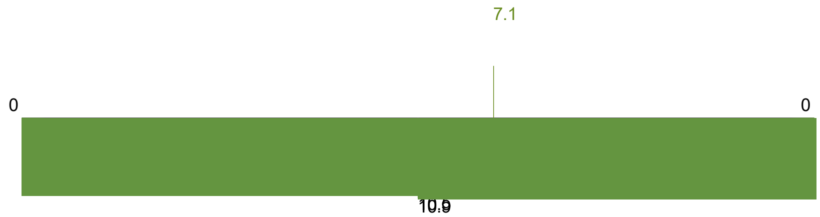
Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.441** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.592** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



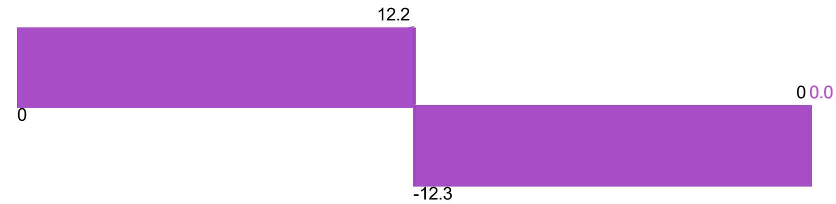
Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **7.15** kip_ft
Tension reinforcement provided	6 No.5 bottom bars (7.0 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **1.86** in^2
Minimum area of reinforcement (10.5.4)	$A_{s.min} = 0.0018 \times L_y \times h$ = **1.814** in^2
	55 5 5 5 5 5
Maximum spacing of reinforcement (10.5.4)	$s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in
	55 5 5 5 5 5 5
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.bot} / 2$ = **20.687** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.782** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.919** in
Strain in tensile reinforcement (10.3.5)	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06450**
	55 5 5 5 5 5
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **188.76** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **169.884** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.042**
	55 5 5 5 5 5 5

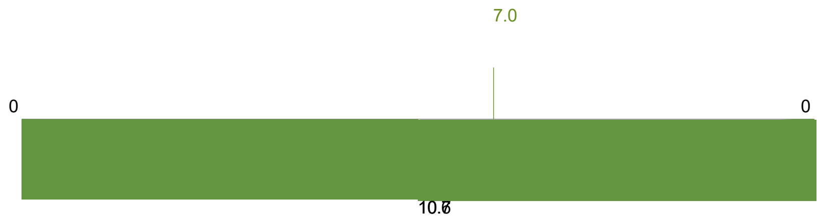
One-way shear design, x direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **7.042** kip_ft

Tension reinforcement provided 6 No.5 bottom bars (7.0 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **1.86** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2

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Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18 \text{ in})$ = **18** in

55 5 5 5 5 5 5

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **20.062** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.782** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.919** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06246**

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Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **182.947** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **164.652** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.043**

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One-way shear design, y direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **20.375** in

Shear perimeter length (11.11.1.2) l_{xp} = **28.375** in

Shear perimeter width (11.11.1.2) l_{yp} = **28.375** in

Shear perimeter (11.11.1.2) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **113.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **805.141** in^2

Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **741.141** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **2.540** ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **13.065** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\ psi) =$ **5.649** psi
Column geometry factor (11.11.2.1)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (11.11.2.1)	$\alpha_s =$ **40**
Concrete shear strength (11.11.2.1)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **580.633** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 11-2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (Eq. 11-1)	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.030**

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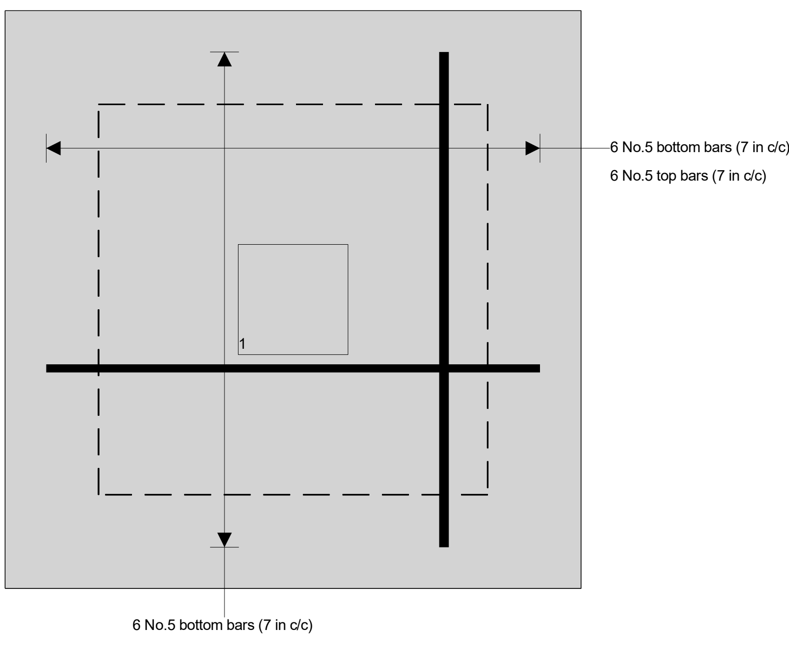
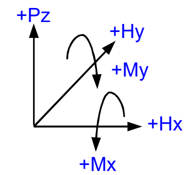


RECTANGULAR SPREAD FOOTING ANALYSIS

For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

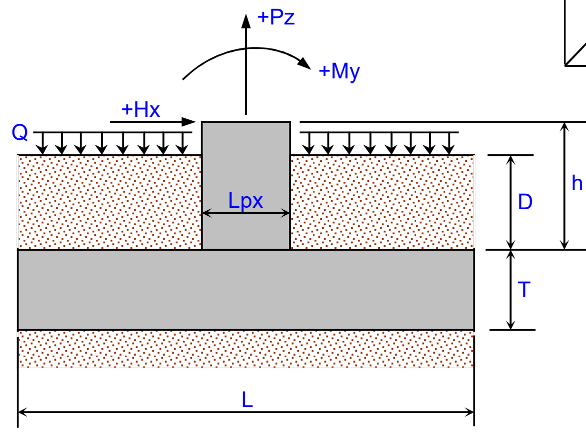
Job Name:	FDOT I-10 Rest Areas	Subject:	Column Footings N-2		
Job Number:	B190988.00	Originator:	EJM	Checker:	



Input Data:

Footing Data:

Footing Length, L =	3.500	ft.
Footing Width, B =	3.500	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

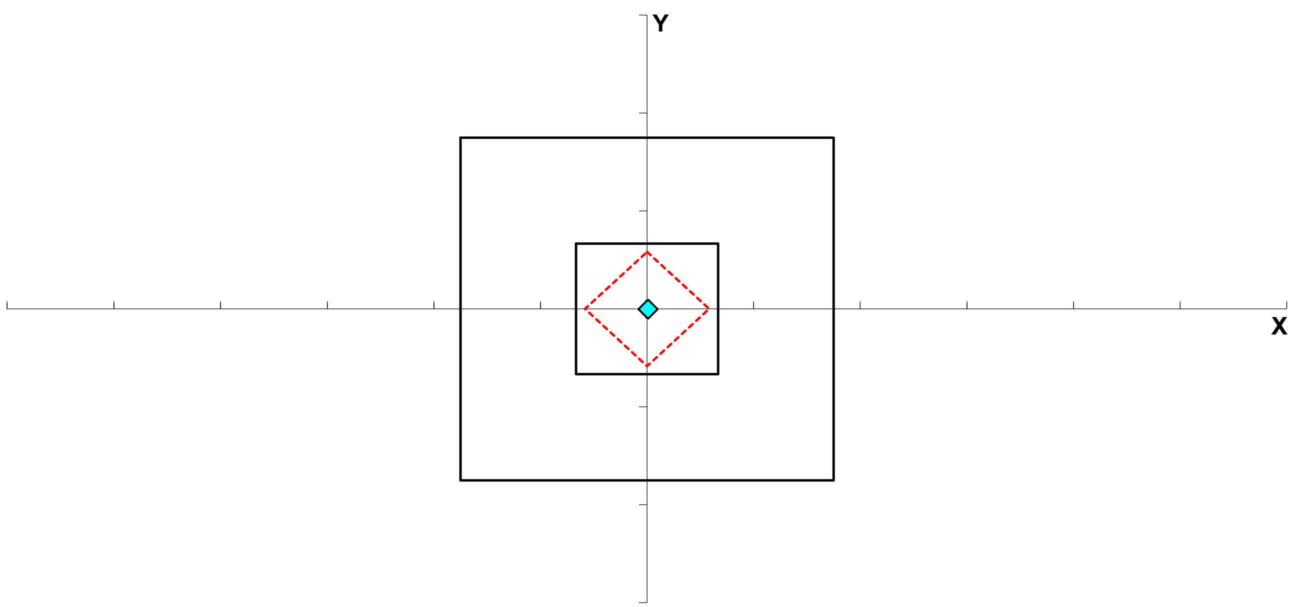


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	1.333							
Lpy (ft.) =	1.333							
h (ft.) =	0.000							
Pz (k) =	-18.11							
Hx (k) =	0.135							
Hy (k) =	0.000							
Mx (ft-k) =	0.000							
My (ft-k) =	0.000							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-23.58	kips
ex =	0.01	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	41.27	ft-kips
ΣMoy =	0.27	ft-kips
FS(ot)y =	152.848	>= 1.5

Sliding Check:

Pass(x) =	5.39	kips
Frict(x) =	9.43	kips
FS(slid)x =	109.793	>= 1.5
Pass(y) =	5.39	kips
Frict(y) =	9.43	kips
FS(slid)y =	N.A.	

Uplift Check:

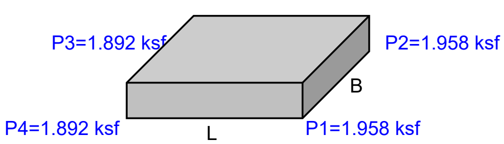
ΣPz(down) =	-23.58	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	3.500	ft.
Brg. Ly =	3.500	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	1.958	ksf
P_2 =	1.958	ksf
P_3 =	1.892	ksf
P_4 =	1.892	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

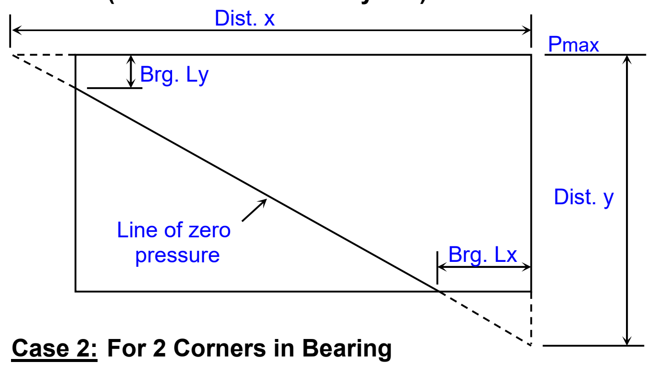
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

$P_{max}(net)$ =	1.591	ksf

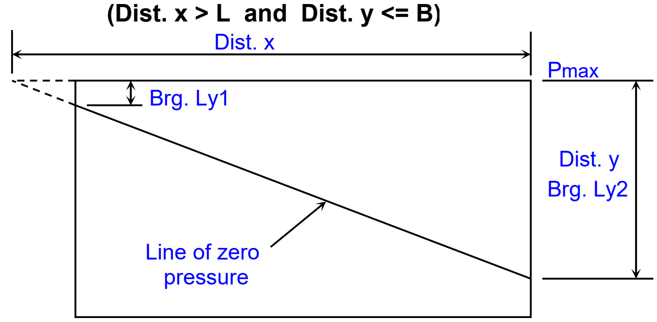
Comments:

Nomenclature for Biaxial Eccentricity:

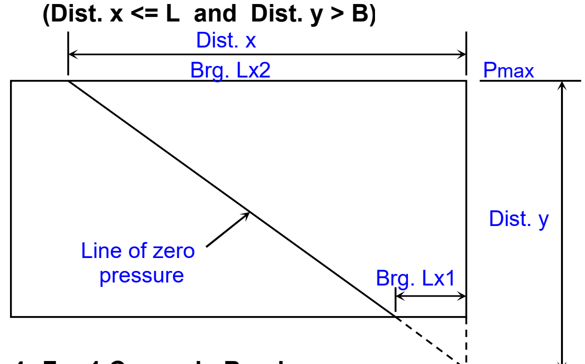
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



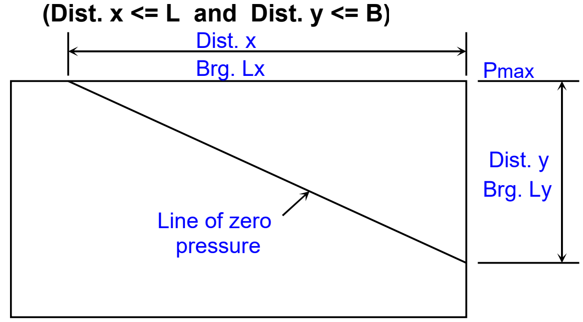
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation L_x = **3.5** ft

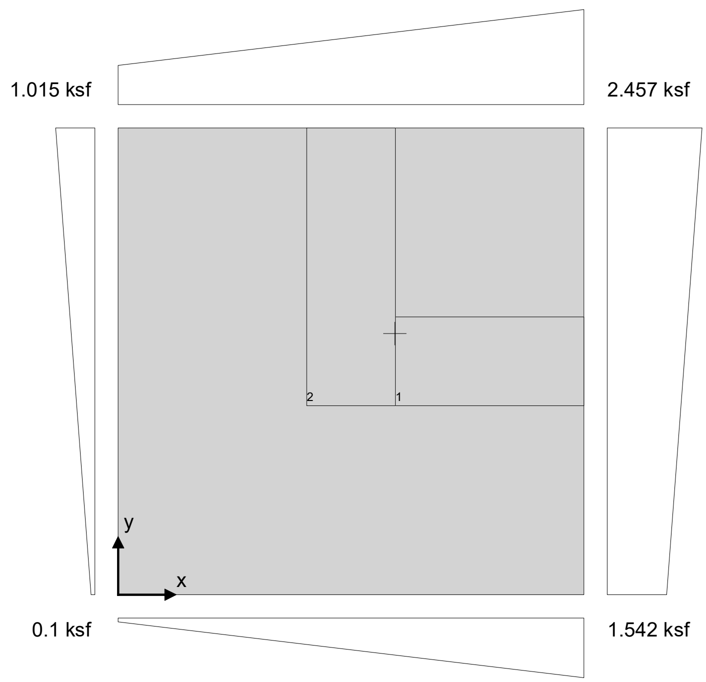
Width of foundation L_y = **3.5** ft

Foundation area $A = L_x \times L_y$ = **12.25** ft^2

Depth of foundation h = **24** in

Depth of soil over foundation h_{soil} = **16** in

Density of concrete γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **17.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **33.50** in

position in y-axis y_1 = **21.00** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **25.00** in

position in x-axis x_2 = **21.00** in

position in y-axis y_2 = **29.50** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2.467** ksf

Density of soil γ_{soil} = **110.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Self weight $F_{swt} = h \times \gamma_{conc}$ = **290** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **146.7** psf

Column no.1 loads

Dead load in z F_{Dz1} = **4.3** kips

Column no.2 loads

Dead load in z F_{Dz2} = **4.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.873)

1.0D + 1.0L (0.873)

1.0D + 1.0Lr (0.873)

1.0D + 1.0S (0.873)

1.0D + 1.0R (0.873)

1.0D + 0.75L + 0.75Lr (0.873)

1.0D + 0.75L + 0.75S (0.873)

1.0D + 0.75L + 0.75R (0.873)

1.0D + 0.6W (0.873)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.996)

1.0D + 0.75L + 0.75Lr + 0.45W (0.873)

1.0D + 0.75L + 0.75S + 0.45W (0.873)

1.0D + 0.75L + 0.75R + 0.45W (0.873)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.961)

0.6D + 0.6W (0.524)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.402)

Combination 10 results: $(1.0 + 0.14 \times S_{DS})$D + 0.7E

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_D \times F_{Dz2}$ = **15.7** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_D \times (F_{Dz2} \times x_2)$ = **32.6** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_D \times (F_{Dz2} \times y_2)$ = **30.7** kip_ft

Uplift verification

Vertical force F_{dz} = **15.658** kips

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{3.948}$ in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{2.505}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.1}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.015}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.542}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.457}$ ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.1}$ ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{2.457}$ ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{2.467}$ ksf

$q_{max} / q_{allow} = \mathbf{0.996}$

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<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-14

Material details

Compressive strength of concrete — $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement — $f_y = \mathbf{60000}$ psi

Cover to reinforcement — $c_{nom} = \mathbf{3}$ in

Concrete type — Normal weight

Concrete modification factor — $\lambda = \mathbf{1.00}$

Column type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.010)
1.2D + 1.6L + 0.5Lr (0.008)
1.2D + 1.6L + 0.5S (0.008)
1.2D + 1.6L + 0.5R (0.008)
1.2D + 1.0L + 1.6Lr (0.008)
1.2D + 1.0L + 1.6S (0.008)
1.2D + 1.0L + 1.6R (0.008)
1.2D + 1.6Lr + 0.5W (0.008)
1.2D + 1.6S + 0.5W (0.008)
1.2D + 1.6R + 0.5W (0.008)
1.2D + 1.0L + 0.5Lr + 1.0W (0.008)
1.2D + 1.0L + 0.5S + 1.0W (0.008)
1.2D + 1.0L + 0.5R + 1.0W (0.008)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.010)
0.9D + 1.0W (0.006)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.005)

Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas				Job Ref. B190988.00	
	Section Wall footer at N-6				Sheet no./rev. 4	
	Calc. by EJM	Date 10/30/2020	Chk'd by	Date	App'd by	Date

Combination 1 results: 1.4D

Forces on foundation

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_D \times F_{Dz2} =$ **19.2** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_D \times (F_{Dz2} \times x_2) =$ **40.0** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_D \times (F_{Dz2} \times y_2) =$ **37.7** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **3.948** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **2.505** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.123** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.246** ksf

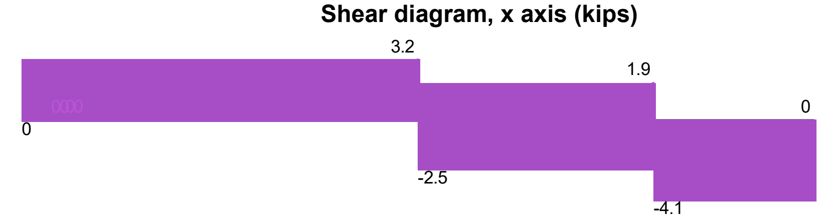
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.893** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.017** ksf

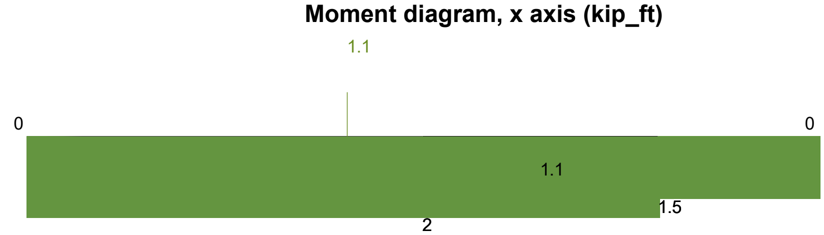
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0.123** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.017** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max} =$ **1.096** kip_ft

Tension reinforcement provided — 5 No.6 bottom bars (8.8 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov} =$ **2.2** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h =$ **1.814** in^2

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18 \text{ in}) =$ **18** in

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2 =$ **19.875** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.924}$ in

Neutral axis factor $\beta_1 = \mathbf{0.85}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{1.087}$ in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05183}$

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{213.541}$ kip_ft

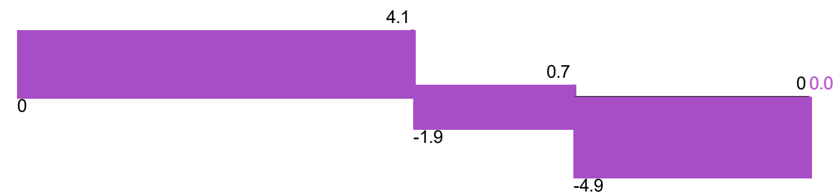
Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity $\phi M_n = \phi_f \times M_n = \mathbf{192.187}$ kip_ft

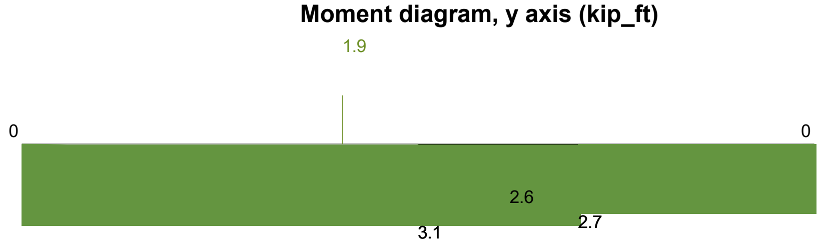
$M_{u.x.max} / \phi M_n = \mathbf{0.006}$

One-way shear design, x direction

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max} = \mathbf{1.926}$ kip_ft

Tension reinforcement provided 5 No.6 bottom bars (8.8 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov} = \mathbf{2.2}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{1.814}$ in^2

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} / 2 = \mathbf{20.625}$ in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.924}$ in

Neutral axis factor $\beta_1 = \mathbf{0.85}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{1.087}$ in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05390}$

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{221.791}$ kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **199.612** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.010**

,*k* , , , , , ,

One-way shear design, y direction

k , , , , , , , , , , , , ,

Two-way shear design at column 1

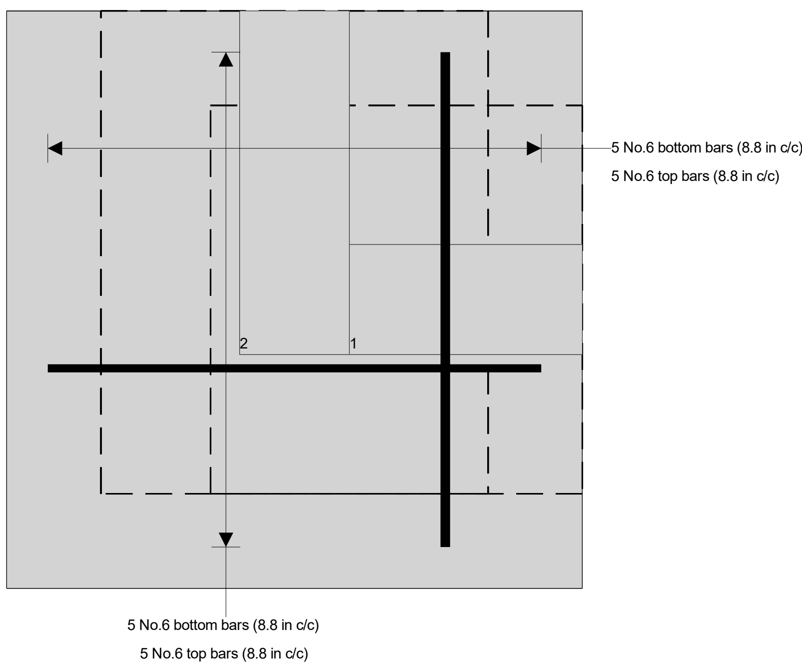
, , *k* , , , , , , , , , , , , , , , , , ,

k , , , , , , , ,

Two-way shear design at column 2

, , *k* , , , , , , , , , , , , , , , , , ,

k , , , , , , , ,



RECTANGULAR SPREAD FOOTING ANALYSIS

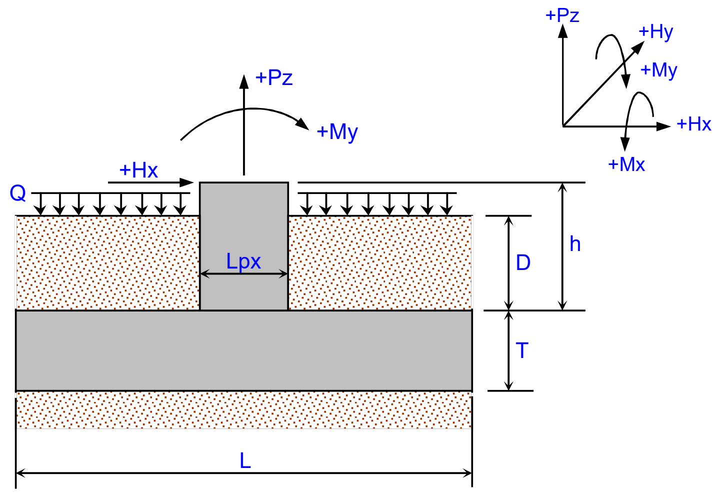
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Col. N-6 Footing		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data:

Footing Data:

Footing Length, L =	3.500	ft.
Footing Width, B =	3.500	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

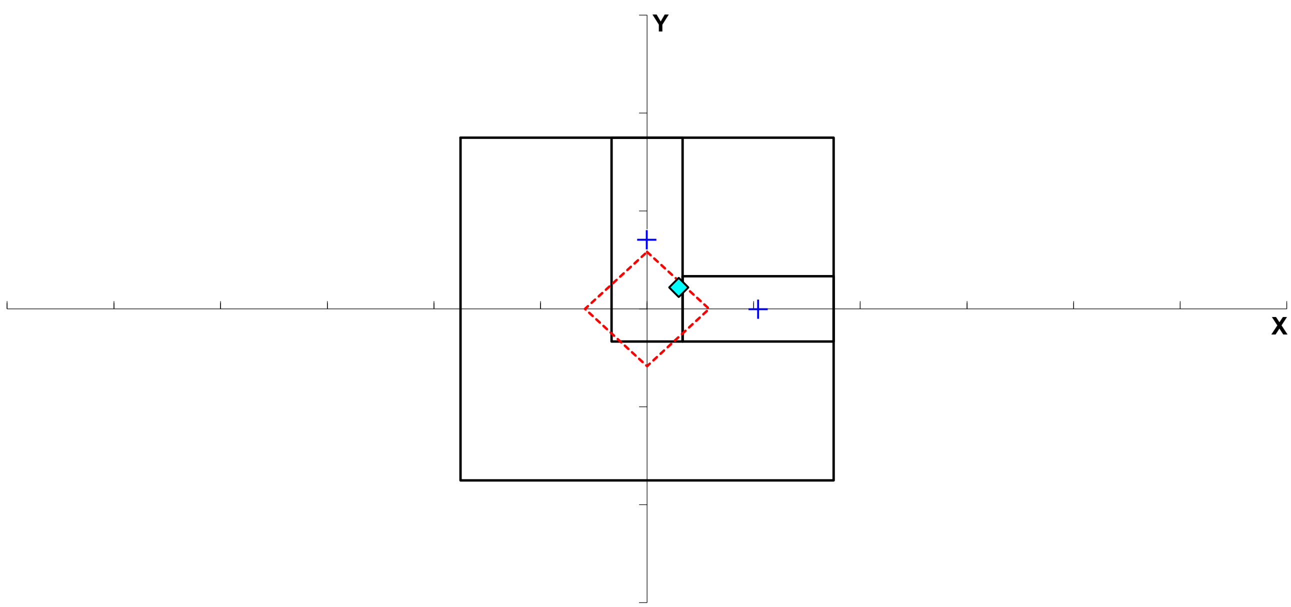


Nomenclature

Pier/Loading Data:

Number of Piers = 2

	Pier #1	Pier #2						
Xp (ft.) =	0.000	1.042						
Yp (ft.) =	0.708	0.000						
Lpx (ft.) =	0.667	1.417						
Lpy (ft.) =	2.083	0.667						
h (ft.) =	0.000	0.000						
Pz (k) =	-4.30	-4.00						
Hx (k) =	0.00	0.00						
Hy (k) =	0.00	0.00						
Mx (ft-k) =	0.00	0.00						
My (ft-k) =	0.00	0.00						



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-13.93	kips
ex =	0.30	ft. (<= L/6)
ey =	0.22	ft. (<= B/6)

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	N.A.	ft-kips
ΣMoy =	N.A.	ft-kips
FS(ot)y =	N.A.	

Sliding Check:

Pass(x) =	5.88	kips
Frict(x) =	5.57	kips
FS(slid)x =	N.A.	
Pass(y) =	5.88	kips
Frict(y) =	5.57	kips
FS(slid)y =	N.A.	

Uplift Check:

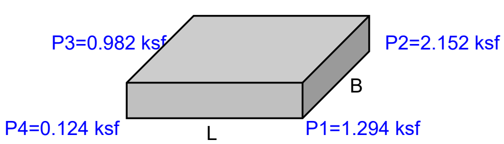
ΣPz(down) =	-13.93	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	3.500	ft.
Brg. Ly =	3.500	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	6*ex/L + 6*ey/B = 0.891

Gross Soil Bearing Corner Pressures:

P_1 =	1.294	ksf
P_2 =	2.152	ksf
P_3 =	0.982	ksf
P_4 =	0.124	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

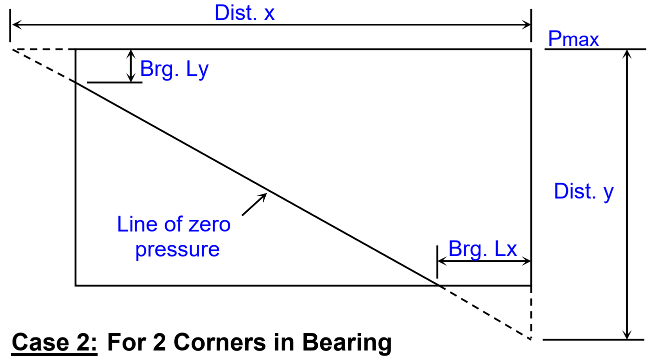
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma_s$

$P_{max}(net)$ =	1.752	ksf

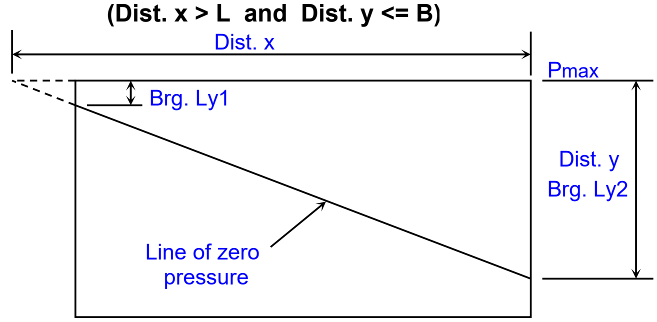
Comments:

Nomenclature for Biaxial Eccentricity:

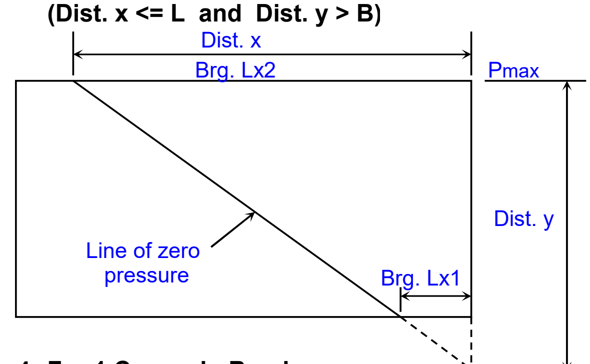
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



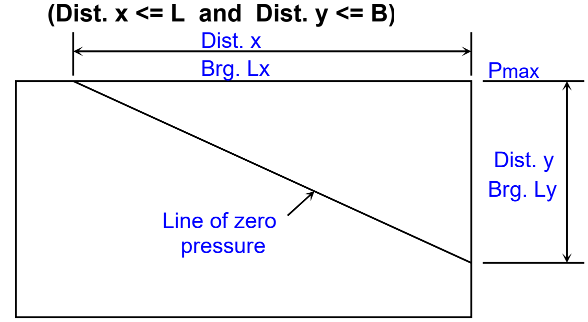
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

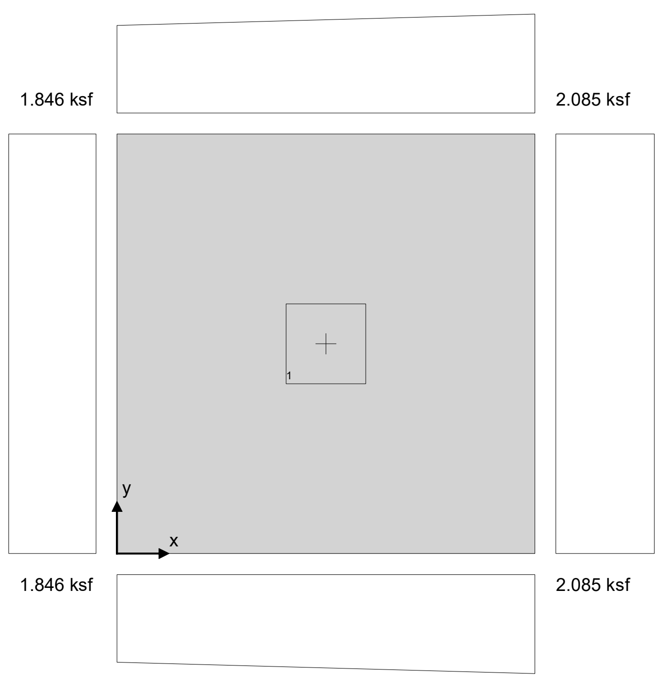


Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **3.5** ft
Width of foundation	L_y = **3.5** ft
Foundation area	$A = L_x \times L_y$ = **12.25** ft^2
Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **8.00** in
Width of column	l_{y1} = **8.00** in
position in x-axis	x_1 = **21.00** in
position in y-axis	y_1 = **21.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.467** ksf
Density of soil	γ_{soil} = **110.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **290** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **146.7** psf

Column no.1 loads

Dead load in z F_{Dz1} = **14.0** kips

Live roof load in z F_{Lrz1} = **4.3** kips

Wind load in z F_{Wz1} = **3.3** kips

Wind load in x F_{Wx1} = **0.3** kips

Wind load moment in x M_{Wx1} = **1.3** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.640)

1.0D + 1.0L (0.640)

1.0D + 1.0Lr (0.783)

1.0D + 1.0S (0.640)

1.0D + 1.0R (0.640)

1.0D + 0.75L + 0.75Lr (0.747)

1.0D + 0.75L + 0.75S (0.640)

1.0D + 0.75L + 0.75R (0.640)

1.0D + 0.6W (0.771)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.729)

1.0D + 0.75L + 0.75Lr + 0.45W (0.845)

1.0D + 0.75L + 0.75S + 0.45W (0.738)

1.0D + 0.75L + 0.75R + 0.45W (0.738)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.704)

0.6D + 0.6W (0.515)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.294)

Combination 11 results: 1.0D + 0.75L + 0.75Lr + 0.45W

Forces on foundation

Force in x-axis $F_{dx} = \gamma_W \times F_{Wx1}$ = **0.1** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **24.1** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1} + F_{Wx1} \times h)$ = **43.0** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **42.1** kip_ft

Uplift verification

Vertical force F_{dz} = **24.08** kips

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Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_W \times (M_{Wx1} + F_{Wx1} \times h)$ = **0.85** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_{Lr} \times (F_{Lrz1} \times (x_1 - L_x)) + \gamma_W \times (F_{Wz1} \times (x_1 - L_x))$ = **-42.14** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL})$ = **49.287**

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Stability against sliding

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb})$ = **13.903** kips

Stability against sliding in x direction

Total sliding resistance $F_{Rx} = F_{RFriction}$ = **13.903** kips

Factor of safety $abs(F_{Rx} / F_{dx})$ = **102.98**

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0.426** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.846** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.846** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.085** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.085** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.846** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.085** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2.467** ksf

q_{max} / q_{allow} = **0.845**

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<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.028)

1.2D + 1.6L + 0.5Lr (0.027)

1.2D + 1.6L + 0.5S (0.024)

1.2D + 1.6L + 0.5R (0.024)

1.2D + 1.0L + 1.6Lr (0.034)

1.2D + 1.0L + 1.6S (0.024)

1.2D + 1.0L + 1.6R (0.024)

1.2D + 1.6Lr + 0.5W (0.036)

1.2D + 1.6S + 0.5W (0.027)

1.2D + 1.6R + 0.5W (0.027)

1.2D + 1.0L + 0.5Lr + 1.0W (0.032)

1.2D + 1.0L + 0.5S + 1.0W (0.029)

1.2D + 1.0L + 0.5R + 1.0W (0.029)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.028)

0.9D + 1.0W (0.023)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.015)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in x-axis — $F_{ux} = \gamma_W \times F_{Wx1}$ = **0.2** kips

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **31.8** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1} + F_{Wx1} \times h)$ = **56.6** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **55.6** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0.359** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.462** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.462** ksf

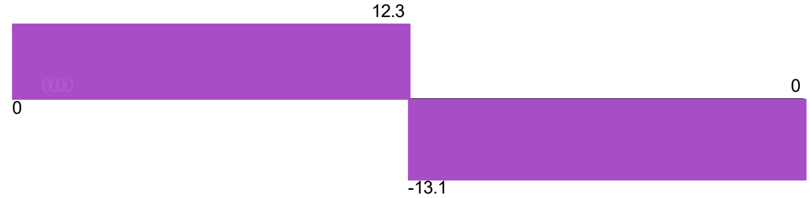
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.728** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.728** ksf

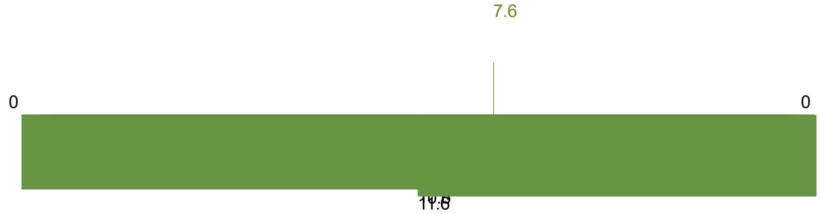
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.462** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.728** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **7.614** kip_ft

Tension reinforcement provided 5 No.6 bottom bars (8.8 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **2.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h$ = **1.814** in^2

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Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

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Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2$ = **20.625** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.924** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.087** in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05390**

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Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **221.791** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **199.612** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.038**

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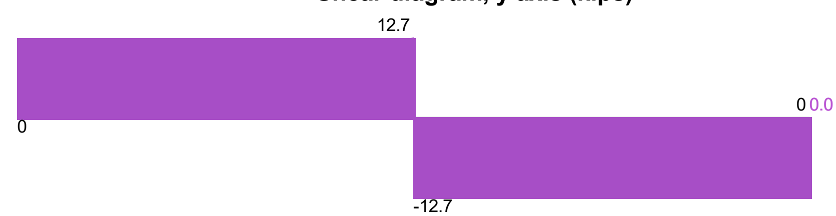
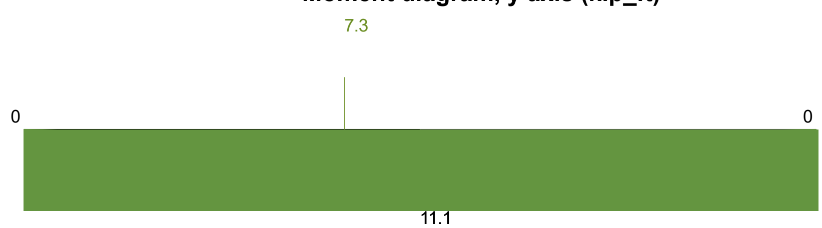
One-way shear design, x direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **7.276** kip_ft

Tension reinforcement provided 5 No.6 bottom bars (8.8 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **2.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2

55 5 5 5 5 5

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 × h, 18 in) = **18** in

55 5 5 5 5 5 5

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **19.875** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.924** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.087** in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05183**

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Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **213.541** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **192.187** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.038**

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One-way shear design, y direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **20.25** in

Shear perimeter length (22.6.4) l_{xp} = **28.250** in

Shear perimeter width (22.6.4) l_{yp} = **28.250** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **113.000** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **798.062** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **734.062** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **2.595** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **13.812** kips

Ultimate shear stress from vertical load v_{ug} = max($F_{up} / (b_o \times d_{v2})$,0 psi) = **6.036** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1}$ = **1.00**

Column location factor (22.6.5.3) α_s =**40**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **579.844** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.032**

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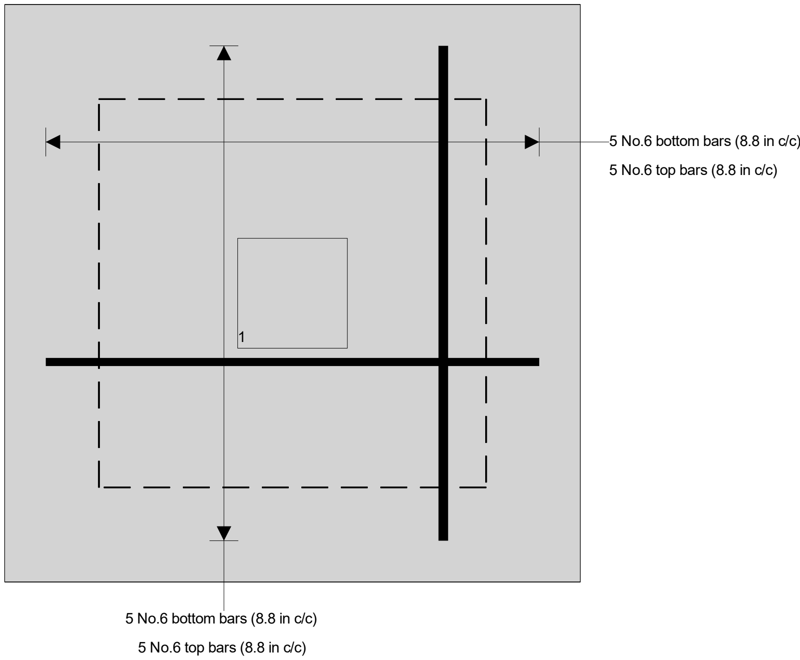


1
5 No.6 bottom bars (8.8 in c/c)
5 No.6 top bars (8.8 in c/c)
5 No.6 bottom bars (8.8 in c/c)
5 No.6 top bars (8.8 in c/c)

RECTANGULAR SPREAD FOOTING ANALYSIS

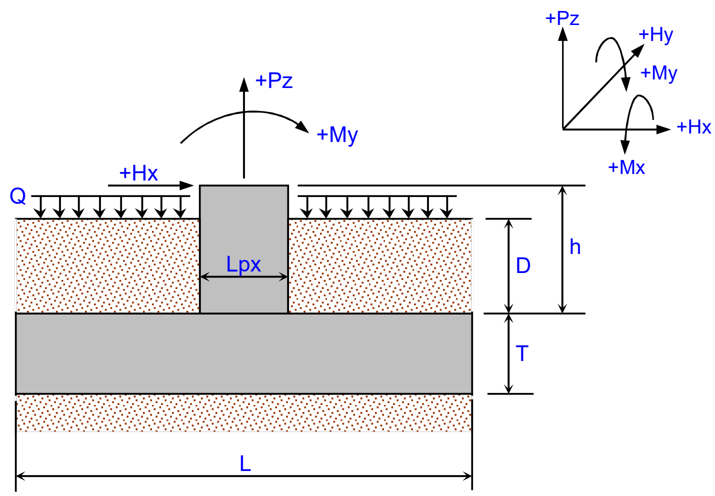
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Col. N-7 Footing	
Job Number:	B190988.00	Originator:	EJM	Checker:

Input Data:

Footing Data:

Footing Length, L =	3.500	ft.
Footing Width, B =	3.500	ft.
Footing Thickness, T =	1.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	2.000	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

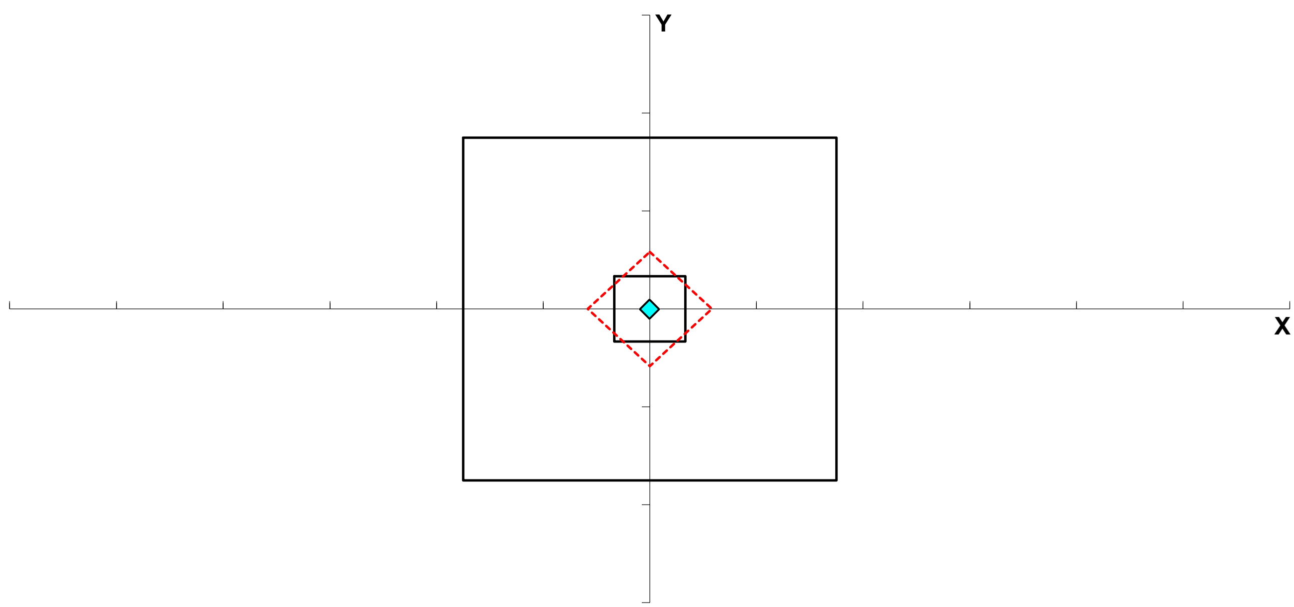


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	0.667							
Lpy (ft.) =	0.667							
h (ft.) =	0.000							
Pz (k) =	-21.65							
Hx (k) =	0.00							
Hy (k) =	0.00							
Mx (ft-k) =	0.00							
My (ft-k) =	0.00							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-26.43	kips
ex =	0.00	
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	N.A.	ft-kips
ΣMoy =	N.A.	ft-kips
FS(ot)y =	N.A.	

Sliding Check:

Pass(x) =	3.15	kips
Frict(x) =	10.57	kips
FS(slid)x =	N.A.	
Pass(y) =	3.15	kips
Frict(y) =	10.57	kips
FS(slid)y =	N.A.	

Uplift Check:

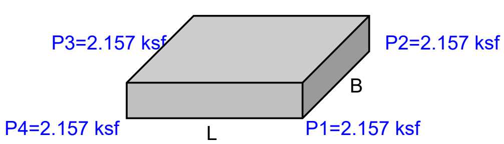
ΣPz(down) =	-26.43	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	3.500	ft.
Brg. Ly =	3.500	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	2.157	ksf
P_2 =	2.157	ksf
P_3 =	2.157	ksf
P_4 =	2.157	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

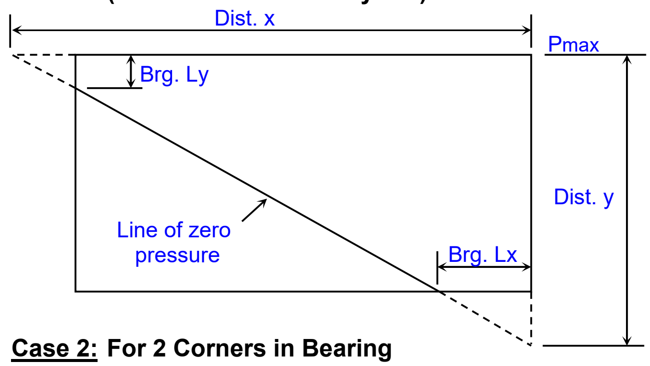
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

$P_{max}(net)$ =	1.797	ksf

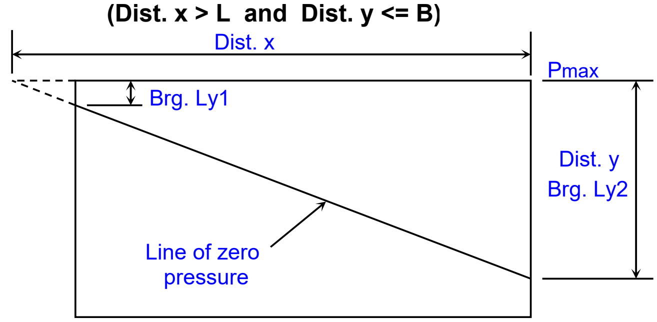
Comments:

Nomenclature for Biaxial Eccentricity:

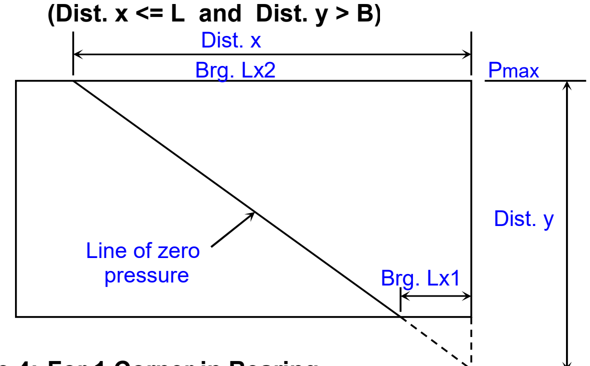
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



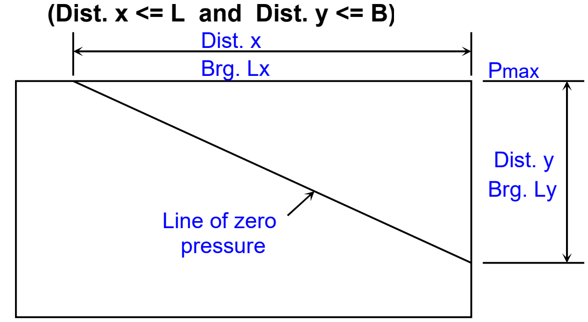
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

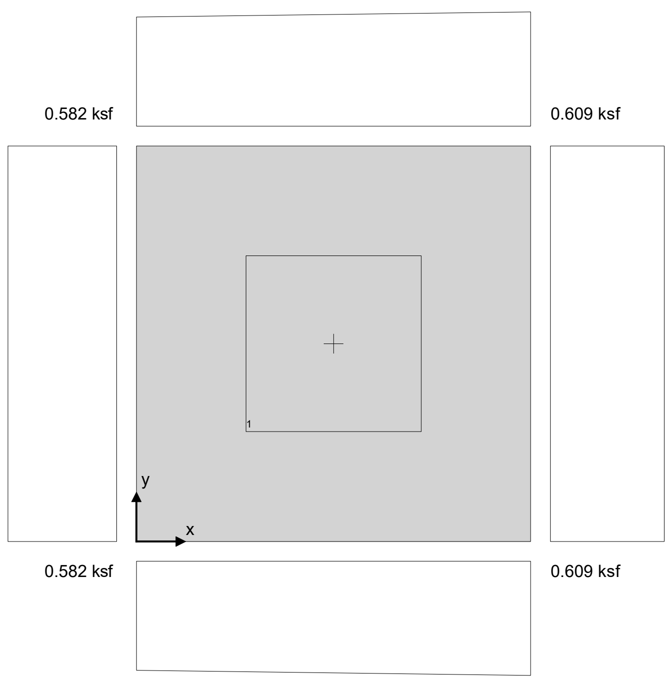


Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **3** ft
Width of foundation	L_y = **3** ft
Foundation area	$A = L_x \times L_y$ = **9** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **16.00** in
Width of column	l_{y1} = **16.00** in
position in x-axis	x_1 = **18.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.1** ksf
Density of soil	γ_{soil} = **110.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil} =$ **146.7** psf

Column no.1 loads

Dead load in z $F_{Dz1} =$ **0.7** kips

Live roof load in z $F_{Lrz1} =$ **1.2** kips

Wind load in z $F_{Wz1} =$ **2.4** kips

Wind load in x $F_{Wx1} =$ **0.1** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.179)

1.0D + 1.0L (0.179)

1.0D + 1.0Lr (0.243)

1.0D + 1.0S (0.179)

1.0D + 1.0R (0.179)

1.0D + 0.75L + 0.75Lr (0.227)

1.0D + 0.75L + 0.75S (0.179)

1.0D + 0.75L + 0.75R (0.179)

1.0D + 0.6W (0.263)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.204)

1.0D + 0.75L + 0.75Lr + 0.45W (0.290)

1.0D + 0.75L + 0.75S + 0.45W (0.242)

1.0D + 0.75L + 0.75R + 0.45W (0.242)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.197)

0.6D + 0.6W (0.192)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.083)

Combination 11 results: 1.0D + 0.75L + 0.75Lr + 0.45W

Forces on foundation

Force in x-axis $F_{dx} = \gamma_W \times F_{Wx1} =$ **0.1** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} =$ **5.4** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + F_{Wx1} \times h) =$ **8.1** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1) =$ **8.0** kip_ft

Uplift verification

Vertical force $F_{dz} =$ **5.359** kips

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Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_W \times (F_{Wx1} \times h) =$ **0.06** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_{Lr} \times (F_{Lrz1} \times (x_1 - L_x)) + \gamma_W \times (F_{Wz1} \times (x_1 - L_x)) =$ **-8.04** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL}) =$ **132.326**

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Stability against sliding

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb}) = \mathbf{3.094}$ kips

Stability against sliding in x direction

Total sliding resistance $F_{Rx} = F_{RFriction} = \mathbf{3.094}$ kips

Factor of safety $abs(F_{Rx} / F_{dx}) = \mathbf{50.93}$

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0.136}$ in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.582}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.582}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.609}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.609}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.582}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{0.609}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{2.1}$ ksf

$q_{max} / q_{allow} = \mathbf{0.290}$

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FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement $f_y = \mathbf{60000}$ psi

Cover to reinforcement $c_{nom} = \mathbf{3}$ in

Concrete type Normal weight

Concrete modification factor $\lambda = \mathbf{1.00}$

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.002)

1.2D + 1.6L + 0.5Lr (0.003)

1.2D + 1.6L + 0.5S (0.002)

1.2D + 1.6L + 0.5R (0.002)

1.2D + 1.0L + 1.6Lr (0.007)

1.2D + 1.0L + 1.6S (0.002)

1.2D + 1.0L + 1.6R (0.002)

1.2D + 1.6Lr + 0.5W (0.011)

1.2D + 1.6S + 0.5W (0.005)

1.2D + 1.6R + 0.5W (0.005)

1.2D + 1.0L + 0.5Lr + 1.0W (0.011)

1.2D + 1.0L + 0.5S + 1.0W (0.009)

1.2D + 1.0L + 0.5R + 1.0W (0.009)

(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.002)

0.9D + 1.0W (0.009)

(0.9 - 0.2 × S_{DS})D + 1.0E (0.001)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in x-axis $F_{ux} = \gamma_W \times F_{Wx1} = \mathbf{0.1}$ kips

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} = \mathbf{7.2}$ kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + F_{Wx1} \times h) = \mathbf{10.8}$ kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1) = \mathbf{10.8}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0.113}$ in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.782}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.782}$ ksf

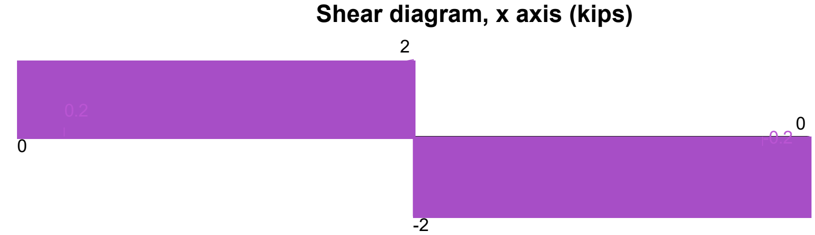
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.812}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.812}$ ksf

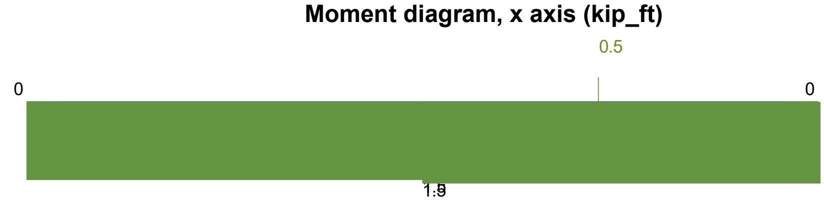
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{0.782}$ ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{0.812}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project FDOT I-10 Rest Areas			Job Ref. B190988.00	
	Section Porch Wood Column Footer			Sheet no./rev. 5	
	Calc. by EJM	Date 10/21/2020	Chk'd by	Date	App'd by	Date

Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max}$ = **0.472** kip_ft

Tension reinforcement provided — 4 No.6 bottom bars (9.7 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov}$ = **1.76** in^2

Minimum area of reinforcement (10.5.4) — $A_{s.min} = 0.0018 \times L_y \times h$ = **0.778** in^2

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Maximum spacing of reinforcement (10.5.4) — $s_{max} = \min(3 \times h, 18 \text{ in})$ = **18** in

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Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} / 2$ = **8.625** in

Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.863** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **1.015** in

Strain in tensile reinforcement (10.3.5) — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02249**

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Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **72.104** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **64.894** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.007**

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One-way shear design, x direction

Ultimate shear force — $V_{u.x}$ = **0.242** kips

Depth to reinforcement — $d_v = h - c_{nom} - \phi_{x.bot} / 2$ = **8.625** in

Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear capacity (Eq. 11-3) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v$ = **39.275** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **29.457** kips

$V_{u.x} / \phi V_n$ = **0.008**

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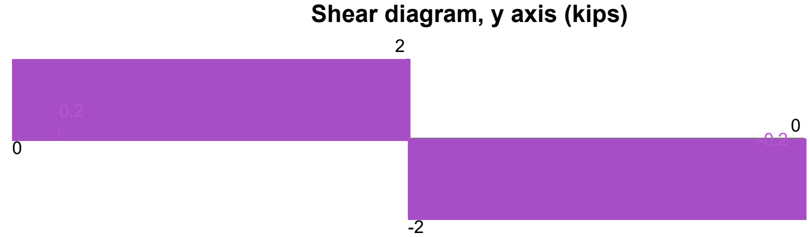
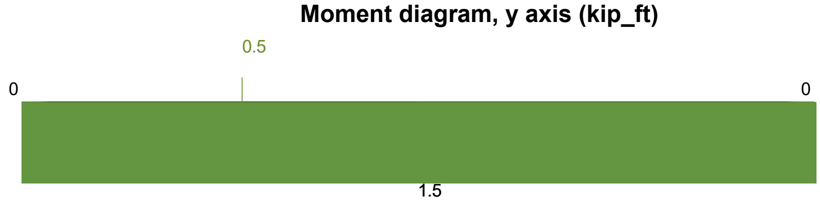
Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **0.46** kip_ft
Tension reinforcement provided	4 No.6 bottom bars (9.7 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **1.76** in^2
Minimum area of reinforcement (10.5.4)	$A_{s.min} = 0.0018 \times L_x \times h$ = **0.778** in^2
	55 5 5 5 5 5
Maximum spacing of reinforcement (10.5.4)	$s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in
	55 5 5 5 5 5 5
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **7.875** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.863** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.015** in
Strain in tensile reinforcement (10.3.5)	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02028**
	55 5 5 5 5 5
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **65.504** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **58.954** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.008**
	55 5 5 5 5 5 5

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **0.235** kips
Depth to reinforcement	$d_v = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **7.875** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 11-3)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **35.86** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **26.895** kips
	$V_{u.y} / \phi V_n$ = **0.009**
	55 5 5 5 5 5 5

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **8.25** in
Shear perimeter length (11.11.1.2)	l_{xp} = **24.250** in
Shear perimeter width (11.11.1.2)	l_{yp} = **24.250** in
Shear perimeter (11.11.1.2)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **97.000** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **588.063** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **332.063** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.797** ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} -$ $q_{up.avg} \times A_p$ = **1.857** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **2.320** psi
Column geometry factor (11.11.2.1)	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (11.11.2.1)	α_s =**40**
Concrete shear strength (11.11.2.1)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **341.656** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 11-2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (Eq. 11-1) $\phi v_n = \phi_v \times v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.012**

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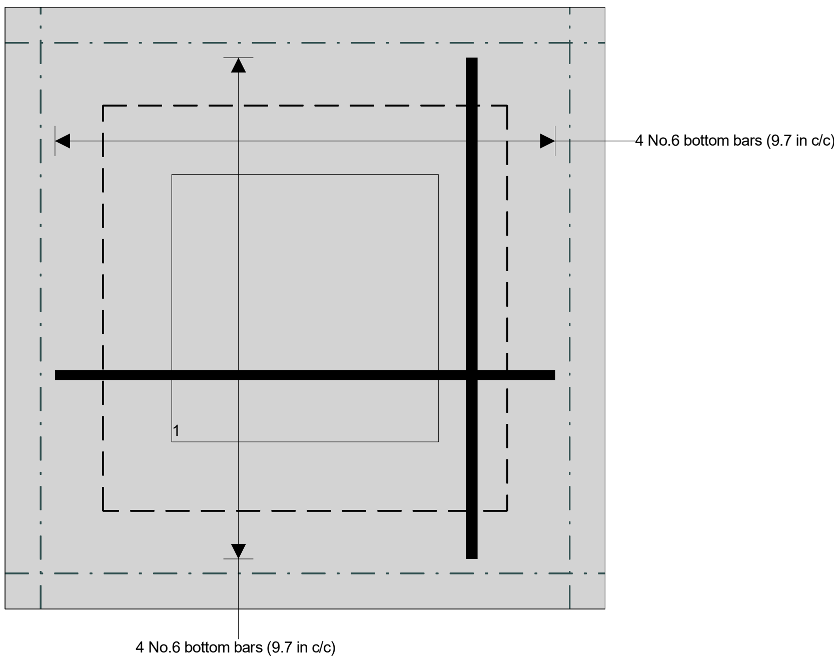


RECTANGULAR SPREAD FOOTING ANALYSIS

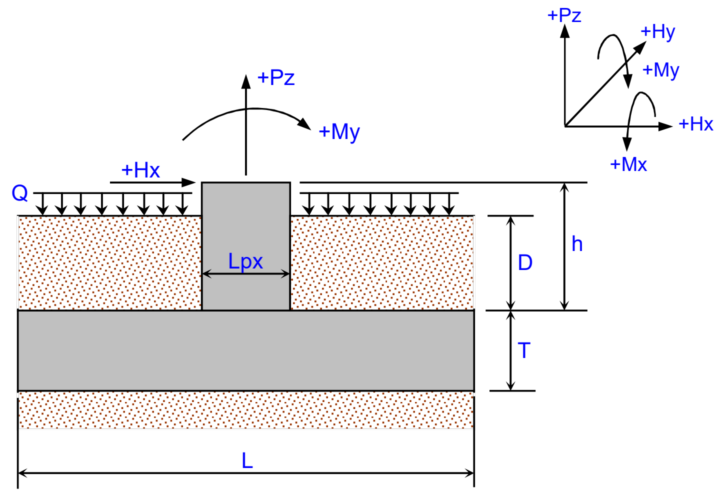
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	FDOT I-10 Rest Areas	Subject:	Wood Column Footings (Porch)		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data:

Footing Data:

Footing Length, L =	3.000	ft.
Footing Width, B =	3.000	ft.
Footing Thickness, T =	1.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

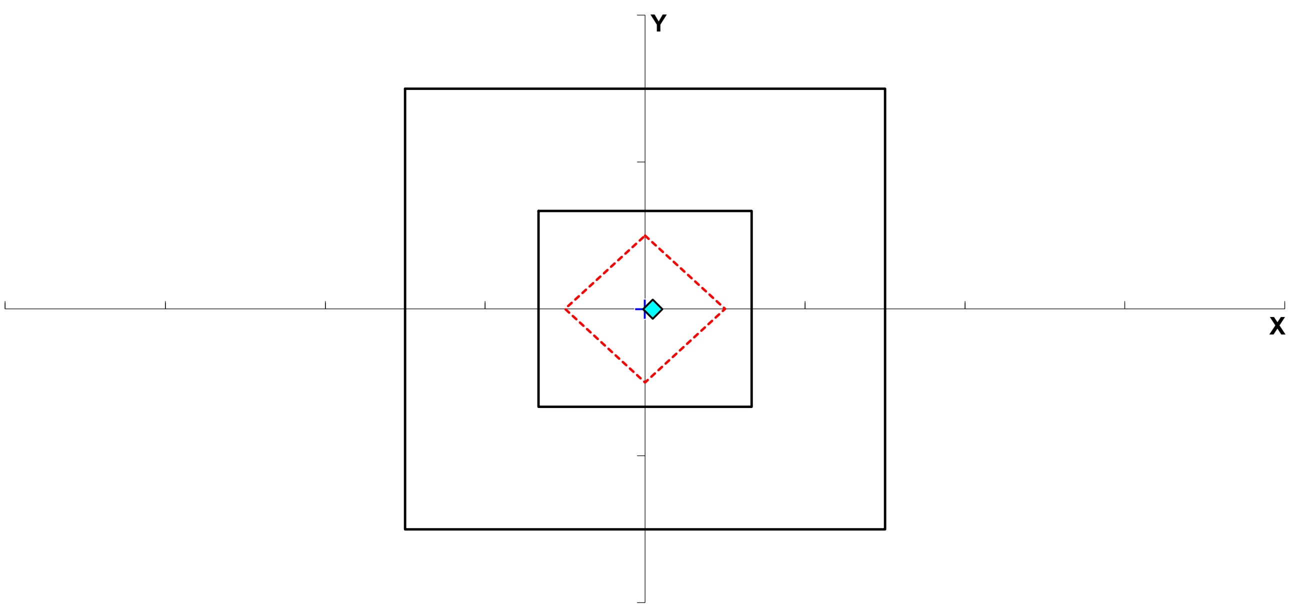


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	1.333							
Lpy (ft.) =	1.333							
h (ft.) =	1.333							
Pz (k) =	-3.36							
Hx (k) =	0.135							
Hy (k) =	0.000							
Mx (ft-k) =	0.000							
My (ft-k) =	0.000							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-6.13	kips
ex =	0.05	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	9.19	ft-kips
ΣMoy =	0.31	ft-kips
FS(ot)y =	29.182	>= 1.5

Sliding Check:

Pass(x) =	1.81	kips
Frict(x) =	2.45	kips
FS(slid)x =	31.597	>= 1.5
Pass(y) =	1.81	kips
Frict(y) =	2.45	kips
FS(slid)y =	N.A.	

Uplift Check:

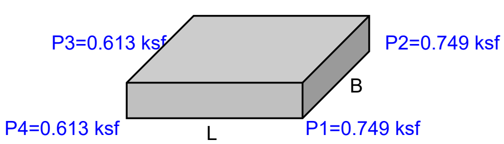
ΣPz(down) =	-6.13	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	3.000	ft.
Brg. Ly =	3.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	0.749	ksf
P_2 =	0.749	ksf
P_3 =	0.613	ksf
P_4 =	0.613	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

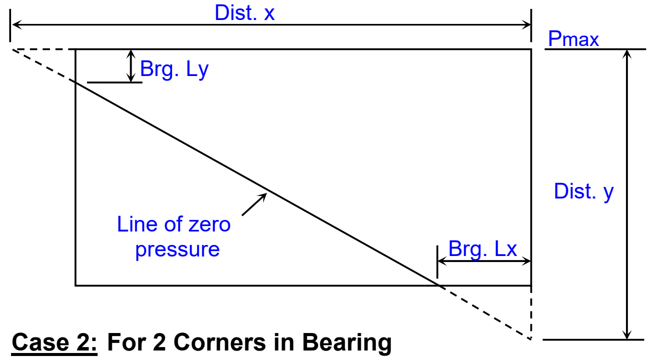
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

P_{max}(net) =	0.492	ksf

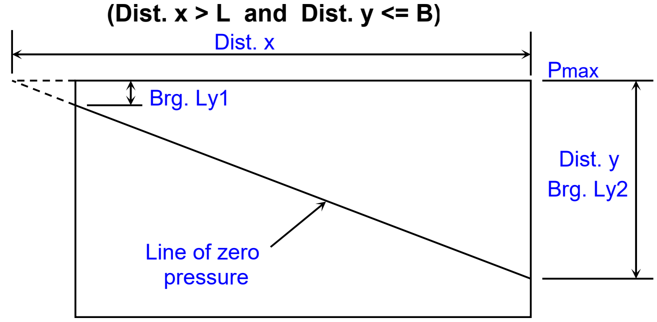
Comments:

Nomenclature for Biaxial Eccentricity:

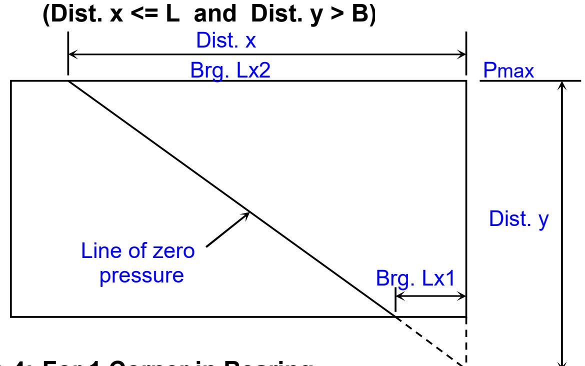
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



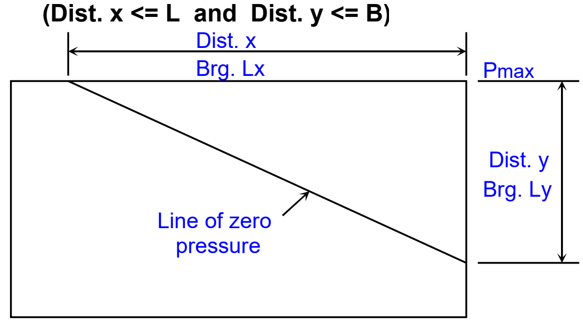
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

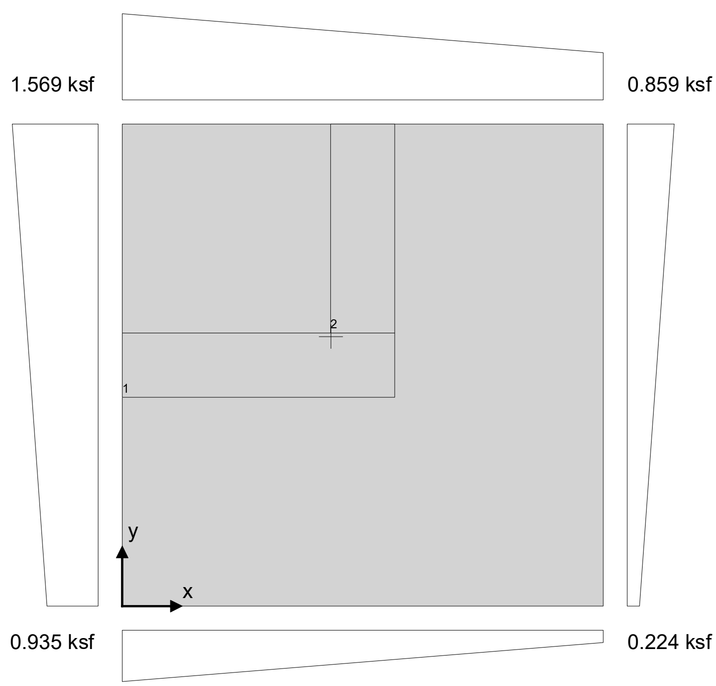


Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **5** ft
Width of foundation	L_y = **5** ft
Foundation area	$A = L_x \times L_y$ = **25** ft^2
Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **34.00** in
Width of column	l_{y1} = **8.00** in
position in x-axis	x_1 = **17.00** in
position in y-axis	y_1 = **30.00** in

Column no.2 details

Length of column	l_{x2} = **8.00** in
Width of column	l_{y2} = **26.00** in
position in x-axis	x_2 = **30.00** in
position in y-axis	y_2 = **47.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.447** ksf

Density of soil γ_{soil} = **110.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction tan(δ_{bb}) = **0.577**

Self weight $F_{swt} = h \times \gamma_{conc}$ = **290** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **146.7** psf

Column no.1 loads

Dead load in z F_{Dz1} = **6.8** kips

Live roof load in z F_{Lrz1} = **4.8** kips

Wind load in z F_{Wz1} = **2.2** kips

Column no.2 loads

Dead load in z F_{Dz2} = **4.7** kips

Live roof load in z F_{Lrz2} = **2.5** kips

Wind load in z F_{Wz2} = **1.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.641)

1.0D + 1.0L (0.641)

Combination 1 results: 1.0D

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_D \times F_{Dz2}$ = **22.4** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_D \times (F_{Dz2} \times x_2)$ = **48.6** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_D \times (F_{Dz2} \times y_2)$ = **62.7** kip_ft

Uplift verification

Vertical force F_{dz} = **22.418** kips

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **-3.963** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **3.539** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.935** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.569** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.224** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.859** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.224** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.569** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2.447** ksf

q_{max} / q_{allow} = **0.641**

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FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.017)

1.2D + 1.6L + 0.5Lr (0.019)

1.2D + 1.6L + 0.5S (0.015)

1.2D + 1.6L + 0.5R (0.015)

1.2D + 1.0L + 1.6Lr (0.029)

1.2D + 1.0L + 1.6S (0.015)

1.2D + 1.0L + 1.6R (0.015)

1.2D + 1.6Lr + 0.5W (0.031)

1.2D + 1.6S + 0.5W (0.017)

1.2D + 1.6R + 0.5W (0.017)

1.2D + 1.0L + 0.5Lr + 1.0W (0.023)

1.2D + 1.0L + 0.5S + 1.0W (0.019)

1.2D + 1.0L + 0.5R + 1.0W (0.019)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.017)

0.9D + 1.0W (0.015)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.009)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times F_{Dz2} + \gamma_{Lr} \times F_{Lrz2} + \gamma_W \times F_{Wz2}$ = **40.5** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1) + \gamma_D \times (F_{Dz2} \times x_2) + \gamma_{Lr} \times (F_{Lrz2} \times x_2) + \gamma_W \times (F_{Wz2} \times x_2)$ = **82.7** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1) + \gamma_D \times (F_{Dz2} \times y_2) + \gamma_{Lr} \times (F_{Lrz2} \times y_2) + \gamma_W \times (F_{Wz2} \times y_2)$ = **115.9** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **-5.477** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **4.338** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.804** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.209** ksf

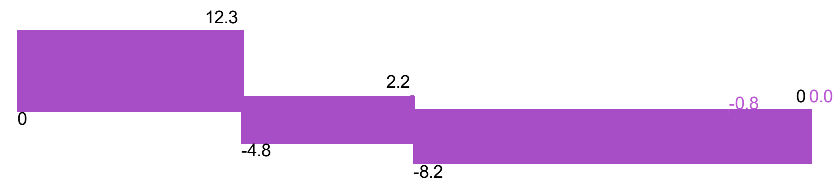
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.03** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.435** ksf

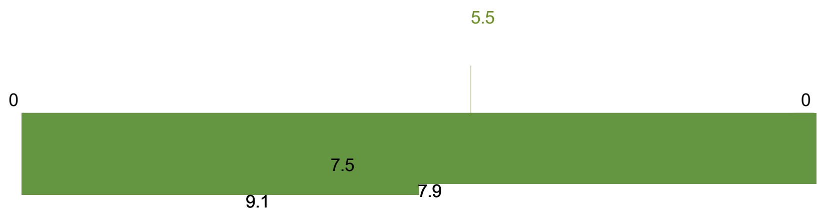
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0.03** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.209** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} =$ **5.456** kip_ft

Tension reinforcement provided 6 No.6 bottom bars (10.6 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} =$ **2.64** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h =$ **2.592** in^2

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18 \text{ in}) =$ **18** in

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2 =$ **19.875** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.776** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.913** in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.06227**

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **257.225** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **231.503** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.024**

Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas				Job Ref. B190988.00	
	Section U-6 Wall Footer				Sheet no./rev. 5	
	Calc. by EJM	Date 10/28/2020	Chk'd by	Date	App'd by	Date

One-way shear design, x direction

Ultimate shear force: $V_{u.x}$ = **0.763** kips

Depth to reinforcement: $d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2)$ = **20.625** in

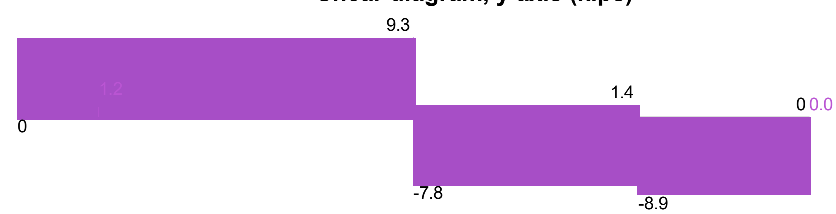
Shear strength reduction factor: ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1): $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v$ = **156.533** kips

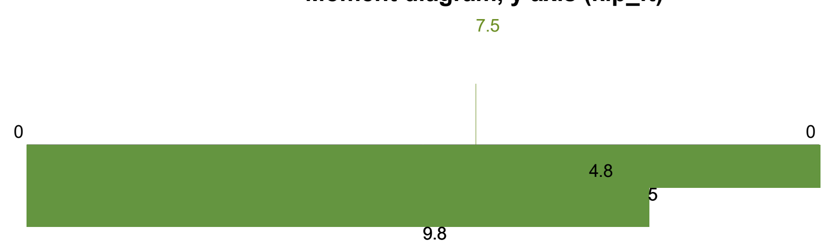
Design shear capacity: $\phi V_n = \phi_v \times V_n$ = **117.4** kips

$V_{u.x} / \phi V_n$ = **0.007**

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment: $M_{u.y.max}$ = **7.528** kip_ft

Tension reinforcement provided: 6 No.6 bottom bars (10.6 in c/c)

Area of tension reinforcement provided: $A_{sy.bot.prov}$ = **2.64** in^2

Minimum area of reinforcement (8.6.1.1): $A_{s.min} = 0.0018 \times L_x \times h$ = **2.592** in^2

Maximum spacing of reinforcement (8.7.2.2): $s_{max} = \min(2 \times h, 18\ in)$ = **18** in

Depth to tension reinforcement: $d = h - c_{nom} - \phi_{y.bot} / 2$ = **20.625** in

Depth of compression block: $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.776** in

Neutral axis factor: β_1 = **0.85**

Depth to neutral axis: $c = a / \beta_1$ = **0.913** in

Strain in tensile reinforcement (8.3.3.1): $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06473**

Nominal moment capacity: $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **267.125** kip_ft

Flexural strength reduction factor: $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity: $\phi M_n = \phi_f \times M_n$ = **240.413** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.031**

One-way shear design, y direction

Ultimate shear force: $V_{u.y}$ = **1.187** kips

Depth to reinforcement: $d_v = \min(h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2)$ = **19.875** in

Shear strength reduction factor: ϕ_v = **0.75**

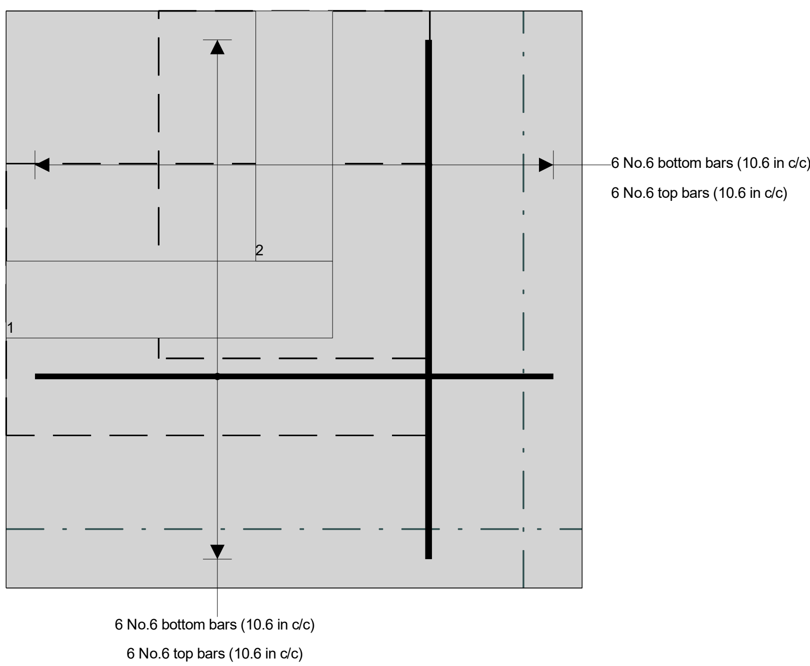
Nominal shear capacity (Eq. 22.5.5.1): $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v$ = **150.841** kips

Design shear capacity: $\phi V_n = \phi_v \times V_n$ = **113.13** kips

$V_{u.y} / \phi V_n$ = **0.010**

Two-way shear design at column 1

Two-way shear design at column 2



RECTANGULAR SPREAD FOOTING ANALYSIS

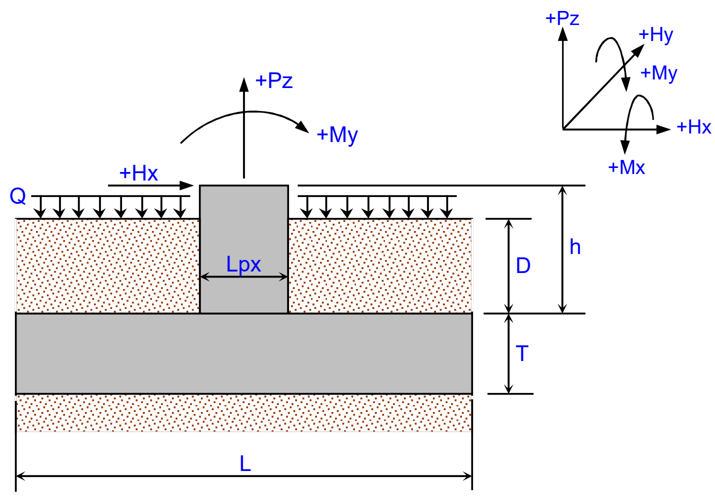
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Porch Footer U-6		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data: Note: Piers represent CMU walls

Footing Data:

Footing Length, L =	5.000	ft.
Footing Width, B =	5.000	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

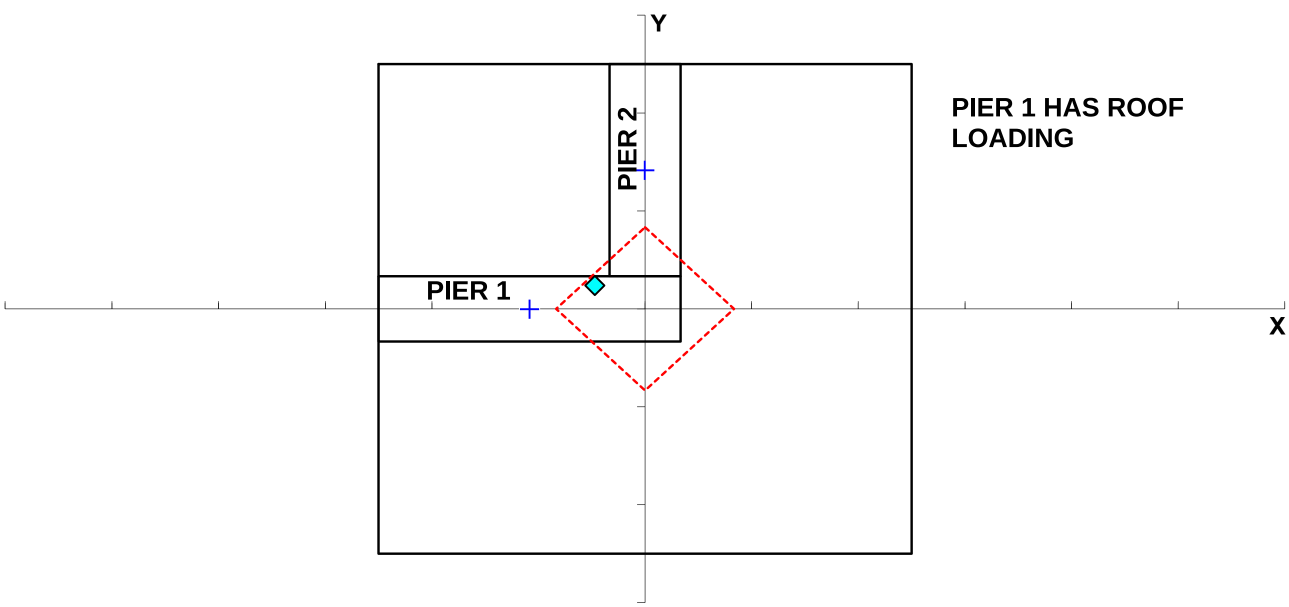


Nomenclature

Pier/Loading Data:

Number of Piers = 2

	Pier #1	Pier #2						
Xp (ft.) =	-1.083	0.000						
Yp (ft.) =	0.000	1.417						
Lpx (ft.) =	2.833	0.667						
Lpy (ft.) =	0.667	2.167						
h (ft.) =	0.000	0.000						
Pz (k) =	-11.706	-4.57						
Hx (k) =	0.000	0.00						
Hy (k) =	0.000	0.00						
Mx (ft-k) =	0.000	0.00						
My (ft-k) =	0.000	0.00						



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-27.19	kips
ex =	-0.47	ft. (<= L/6)
ey =	0.24	ft. (<= B/6)

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	N.A.	ft-kips
ΣMoy =	N.A.	ft-kips
FS(ot)y =	N.A.	

Sliding Check:

Pass(x) =	7.70	kips
Frict(x) =	10.88	kips
FS(slid)x =	N.A.	
Pass(y) =	7.70	kips
Frict(y) =	10.88	kips
FS(slid)y =	N.A.	

Uplift Check:

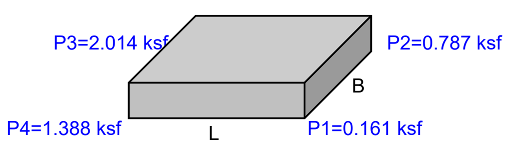
ΣPz(down) =	-27.19	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	5.000	ft.
Brg. Ly =	5.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	6*ex/L + 6*ey/B = 0.852

Gross Soil Bearing Corner Pressures:

P_1 =	0.161	ksf
P_2 =	0.787	ksf
P_3 =	2.014	ksf
P_4 =	1.388	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma_s$

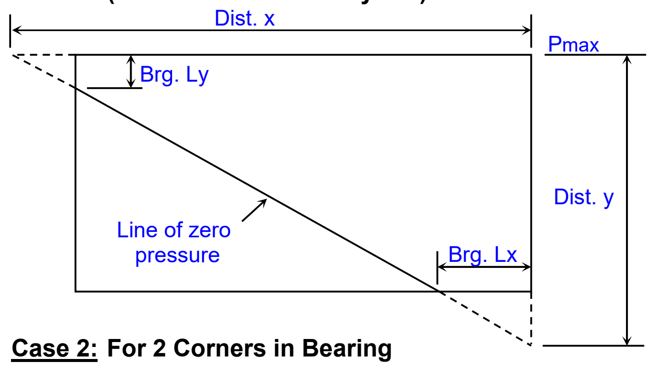
P_{max}(net) =	1.648	ksf

Comments:

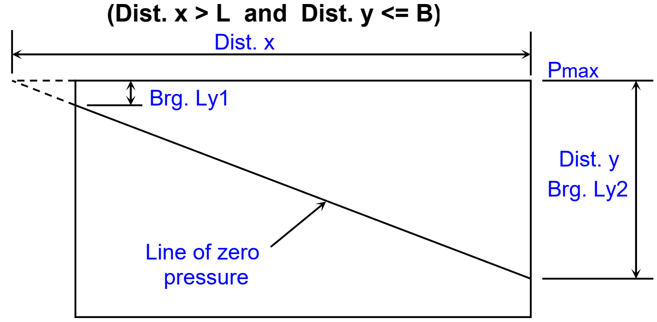
Beam RB6 reaction is spread ovet length of walls

Nomenclature for Biaxial Eccentricity:

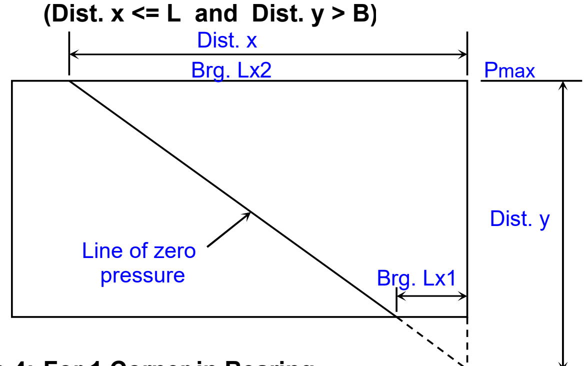
Case 1: For 3 Corners in Bearing (Dist. x > L and Dist. y > B)



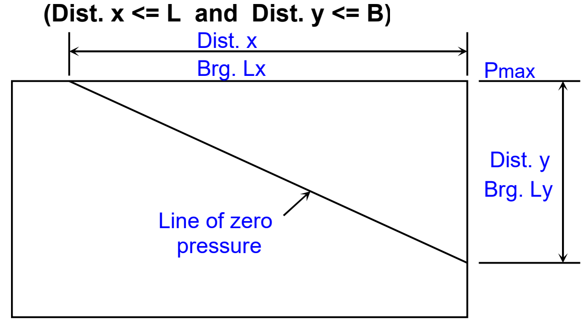
Case 2: For 2 Corners in Bearing (Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing (Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing (Dist. x <= L and Dist. y <= B)



Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation L_x = **3.5** ft

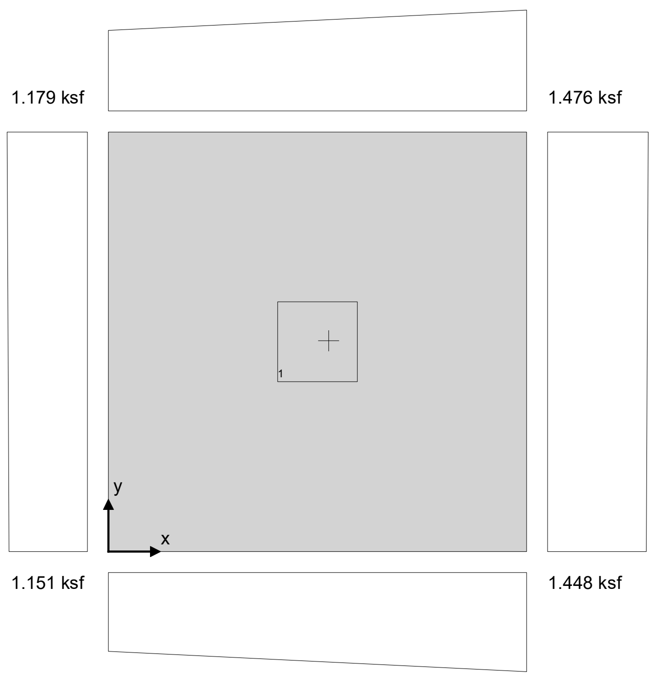
Width of foundation L_y = **3.5** ft

Foundation area $A = L_x \times L_y$ = **12.25** ft^2

Depth of foundation h = **24** in

Depth of soil over foundation h_{soil} = **16** in

Density of concrete γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **21.00** in

position in y-axis y_1 = **21.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2.1** ksf

Density of soil γ_{soil} = **110.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Passive pressure coefficient (Rankine) $K_P = (1 + \sin(\phi_b)) / (1 - \sin(\phi_b))$ = **3**

Self weight $F_{swt} = h \times \gamma_{conc} =$ **290** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil} =$ **146.7** psf

Column no.1 loads

Dead load in z $F_{Dz1} =$ **6.1** kips

Live roof load in z $F_{Lrz1} =$ **4.7** kips

Dead load in x $F_{Dx1} =$ **0.5** kips

Live load in x $F_{Lx1} =$ **0.1** kips

Wind load in y $F_{Wy1} =$ **0.3** kips

Dead load moment in x $M_{Dx1} =$ **0.1** kip_ft

Live load moment in x $M_{Lx1} =$ **0.1** kip_ft

Dead load moment in y $M_{Dy1} =$ **0.1** kip_ft

Live load moment in y $M_{Ly1} =$ **0.1** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.522)

1.0D + 1.0L (0.551)

1.0D + 1.0Lr (0.703)

1.0D + 1.0S (0.522)

1.0D + 1.0R (0.522)

1.0D + 0.75L + 0.75Lr (0.679)

1.0D + 0.75L + 0.75S (0.544)

1.0D + 0.75L + 0.75R (0.544)

1.0D + 0.6W (0.546)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.595)

1.0D + 0.75L + 0.75Lr + 0.45W (0.697)

1.0D + 0.75L + 0.75S + 0.45W (0.562)

1.0D + 0.75L + 0.75R + 0.45W (0.562)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.596)

0.6D + 0.6W (0.337)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.240)

Combination 3 results: 1.0D + 1.0Lr

Forces on foundation

Force in x-axis $F_{dx} = \gamma_D \times F_{Dx1} =$ **0.5** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} =$ **16.1** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) =$ **29.2** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1 + M_{Dy1}) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) =$ **28.3** kip_ft

Uplift verification

Vertical force $F_{dz} =$ **16.089** kips

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Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_D \times (M_{Dx1}+F_{Dx1} \times h) = \mathbf{1.06}$ kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_{Lr} \times (F_{Lrz1} \times (x_1 - L_x)) = \mathbf{-28.16}$ kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL}) = \mathbf{26.562}$

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Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_D \times (M_{Dy1}) = \mathbf{0.1}$ kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (F_{Dz1} \times (y_1 - L_y)) + \gamma_{Lr} \times (F_{Lrz1} \times (y_1 - L_y)) = \mathbf{-28.16}$ kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL}) = \mathbf{281.560}$

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Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 \text{ kN}) \times tan(\delta_{bb}) = \mathbf{9.289}$ kips

Stability against sliding in x direction

Resistance from passive soil pressure $F_{RxPass} = 0.5 \times K_P \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil} = \mathbf{5.39}$ kips

Total sliding resistance $F_{Rx} = F_{RFriction} + F_{RxPass} = \mathbf{14.679}$ kips

Factor of safety $abs(F_{Rx} / F_{dx}) = \mathbf{30.58}$

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0.791}$ in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0.075}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.151}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.179}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.448}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.476}$ ksf

Minimum base pressure $q_{min} = min(q_1,q_2,q_3,q_4) = \mathbf{1.151}$ ksf

Maximum base pressure $q_{max} = max(q_1,q_2,q_3,q_4) = \mathbf{1.476}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{2.1}$ ksf

$q_{max} / q_{allow} = \mathbf{0.703}$

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<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement $f_y = \mathbf{60000}$ psi

Cover to reinforcement $c_{nom} = \mathbf{3}$ in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.015)

1.2D + 1.6L + 0.5Lr (0.017)

1.2D + 1.6L + 0.5S (0.013)

1.2D + 1.6L + 0.5R (0.013)

1.2D + 1.0L + 1.6Lr (0.025)

1.2D + 1.0L + 1.6S (0.013)

1.2D + 1.0L + 1.6R (0.013)

1.2D + 1.6Lr + 0.5W (0.026)

1.2D + 1.6S + 0.5W (0.013)

1.2D + 1.6R + 0.5W (0.013)

1.2D + 1.0L + 0.5Lr + 1.0W (0.018)

1.2D + 1.0L + 0.5S + 1.0W (0.014)

1.2D + 1.0L + 0.5R + 1.0W (0.014)

$(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.2S + 1.0E$ (0.015)

0.9D + 1.0W (0.011)

$(0.9 - 0.2 \times S_{DS})D + 1.0E$ (0.008)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in x-axis $F_{ux} = \gamma_D \times F_{Dx1}$ = **0.6** kips

Ultimate force in y-axis $F_{uy} = \gamma_W \times F_{Wy1}$ = **0.2** kips

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1}$ = **21.2** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **38.3** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1 + M_{Dy1}) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wy1} \times h)$ = **37.5** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0.721** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.238** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.491** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.609** ksf

$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.847** ksf

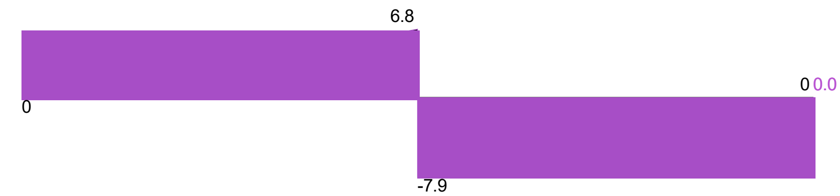
$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.965** ksf

Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.491** ksf

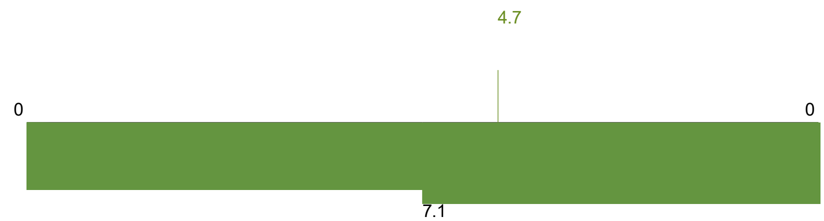
Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.965** ksf

Tekla Tedds GAI Consultants 600 Cranberry Woods Drive Suite 400 Cranberry Township, PA. 16066	Project I-10 Rest Areas			Job Ref. B190988.00	
	Section Porch Column V-3 Footer			Sheet no./rev. 5	
	Calc. by EJM	Date 10/20/2020	Chk'd by	Date	App'd by	Date

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **4.686** kip_ft

Tension reinforcement provided 6 No.5 bottom bars (7.0 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **1.86** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_y \times h$ = **1.814** in^2

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Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in

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Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2$ = **20.687** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.782** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.919** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06450**

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Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **188.76** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **169.884** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.028**

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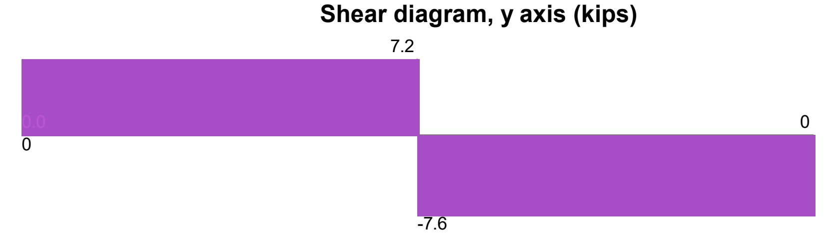
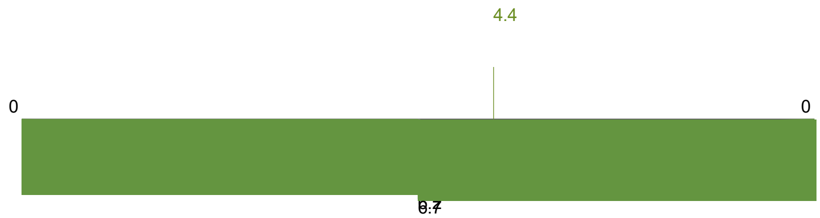
One-way shear design, x direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **4.38** kip_ft

Tension reinforcement provided 6 No.5 bottom bars (7.0 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **1.86** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2

55 5 5 5 5 5

Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in

55 5 5 5 5 5 5

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **20.062** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.782** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.919** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06246**

55 5 5 5 5 5

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **182.947** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **164.652** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.027**

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One-way shear design, y direction

5 5 5 5 5 5 5 5 5 5 5 5 5

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **20.375** in

Shear perimeter length (11.11.1.2) l_{xp} = **28.375** in

Shear perimeter width (11.11.1.2) l_{yp} = **28.375** in

Shear perimeter (11.11.1.2) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **113.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **805.141** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **741.141** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **1.808** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **7.496** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **3.242** psi

Column geometry factor (11.11.2.1) $\beta = l_{y1} / l_{x1}$ = **1.00**

Column location factor (11.11.2.1) α_s =**40**

Concrete shear strength (11.11.2.1) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **580.633** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 11-2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (Eq. 11-1) $\phi v_n = \phi_v \times v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.017**

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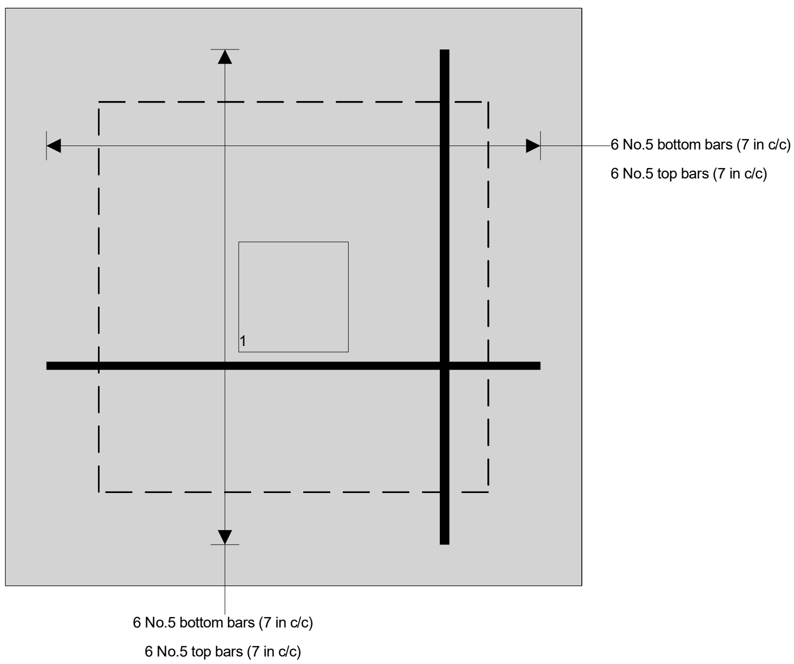


RECTANGULAR SPREAD FOOTING ANALYSIS

For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Porch Footer V-3		
Job Number:	B190988.00	Originator:	EJM	Checker:	

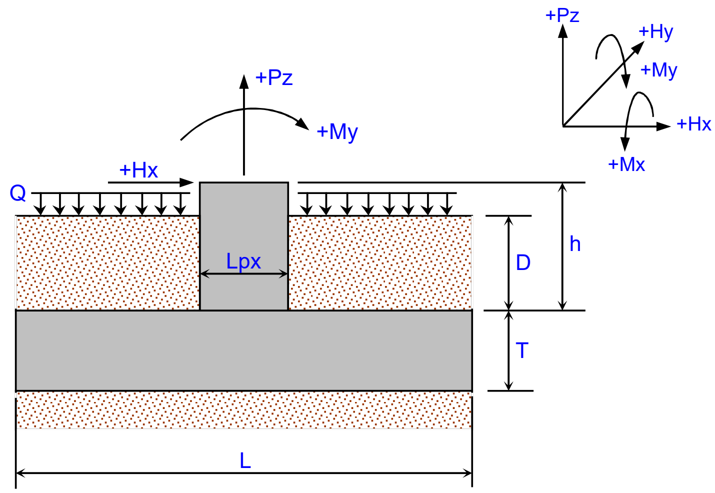
Input Data: LC 104 ASD for soil bearing

Footing Data:

Footing Length, L =	3.500	ft.
Footing Width, B =	3.500	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

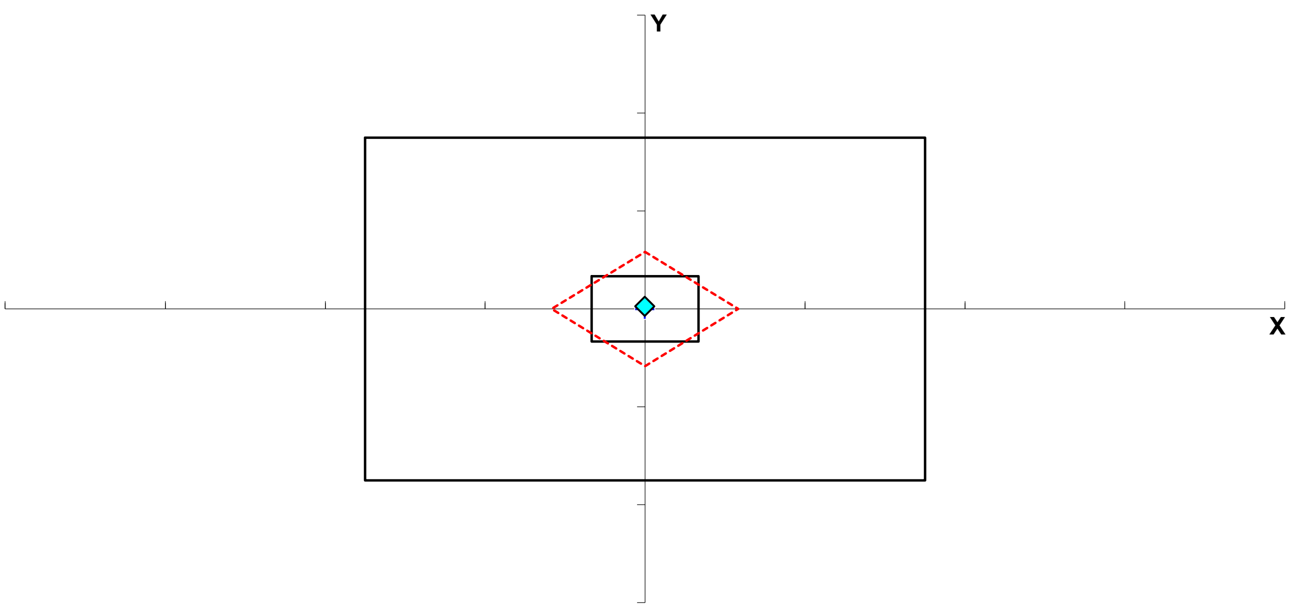
Pier/Loading Data:

Number of Piers = 1



Nomenclature

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	0.667							
Lpy (ft.) =	0.667							
h (ft.) =	0.000							
Pz (k) =	-15.241							
Hx (k) =	0.000							
Hy (k) =	0.300							
Mx (ft-k) =	0.000							
My (ft-k) =	0.003							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-20.59	kips
ex =	0.00	
ey =	0.03	ft. (<= B/6)

Overturning Check:

ΣMrx =	36.03	ft-kips
ΣMox =	-0.60	ft-kips
FS(ot)x =	60.053	>= 1.5
ΣMry =	N.A.	ft-kips
ΣMoy =	N.A.	ft-kips
FS(ot)y =	N.A.	>= 1.5

Sliding Check:

Pass(x) =	5.39	kips
Frict(x) =	8.24	kips
FS(slid)x =	N.A.	
Pass(y) =	5.39	kips
Frict(y) =	8.24	kips
FS(slid)y =	45.417	>= 1.5

Uplift Check:

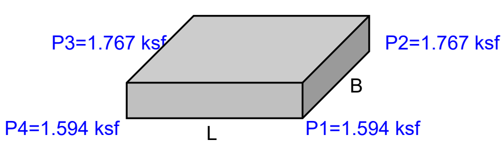
ΣPz(down) =	-20.59	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	3.500	ft.
Brg. Ly =	3.500	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	1.594	ksf
P_2 =	1.767	ksf
P_3 =	1.767	ksf
P_4 =	1.594	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

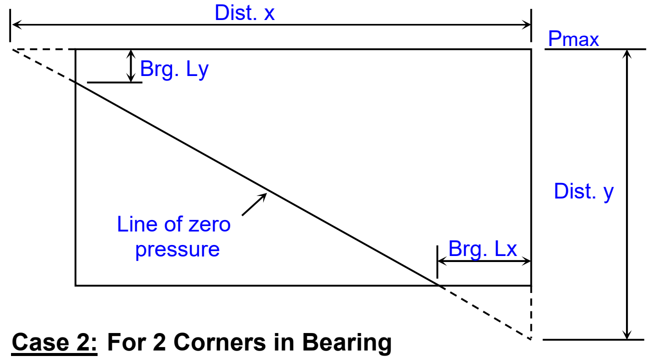
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

$P_{max}(net)$ =	1.401	ksf

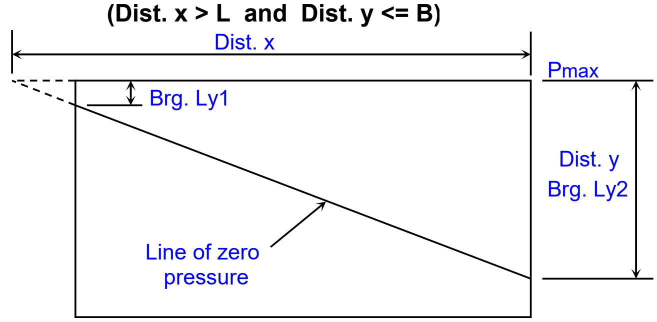
Comments:

Nomenclature for Biaxial Eccentricity:

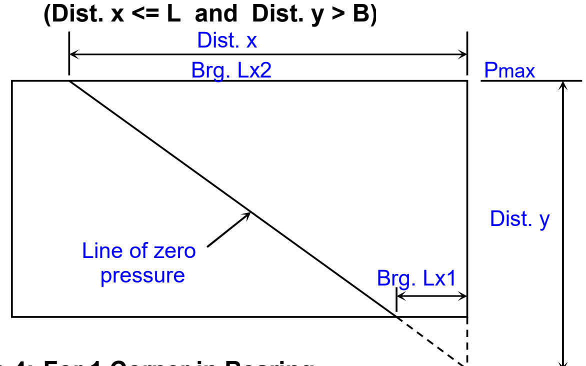
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



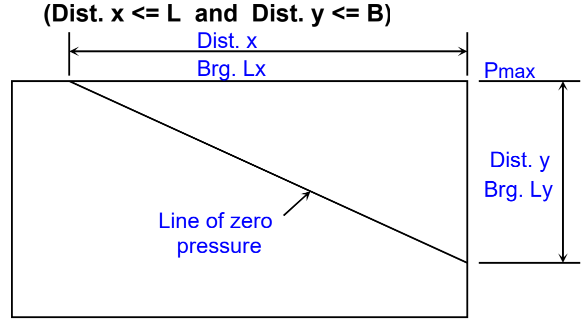
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



Foundation analysis & design (ACI318) in accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation L_x = **5** ft

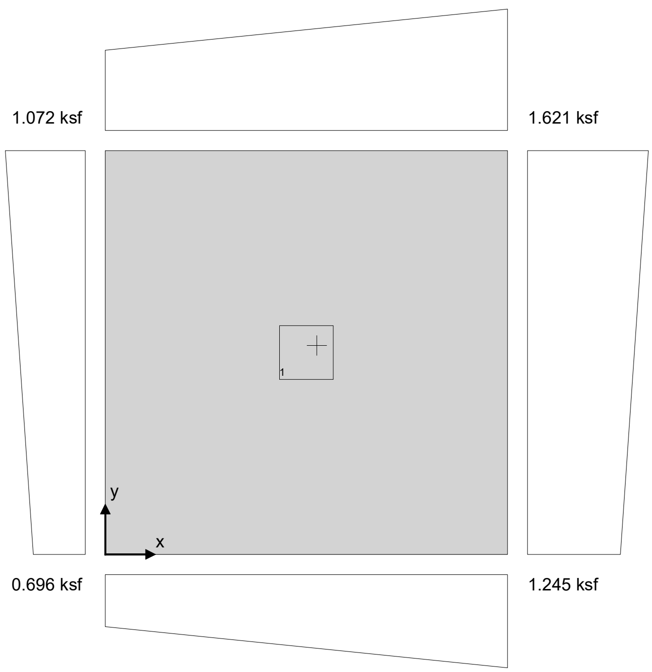
Width of foundation L_y = **5** ft

Foundation area $A = L_x \times L_y$ = **25** ft^2

Depth of foundation h = **18** in

Depth of soil over foundation h_{soil} = **16** in

Density of concrete γ_{conc} = **145.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **30.00** in

position in y-axis y_1 = **30.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2.1** ksf

Density of soil γ_{soil} = **110.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Passive pressure coefficient (Rankine) $K_P = (1 + \sin(\phi_b)) / (1 - \sin(\phi_b))$ = **3**

Foundation loads

Self weight $F_{swt} = h \times \gamma_{conc} = \mathbf{217.5}$ psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil} = \mathbf{146.7}$ psf

Column no.1 loads

Dead load in z $F_{Dz1} = \mathbf{10.0}$ kips

Live load in z $F_{Lz1} = \mathbf{9.9}$ kips

Dead load in x $F_{Dx1} = \mathbf{0.3}$ kips

Live load in x $F_{Lx1} = \mathbf{0.4}$ kips

Dead load in y $F_{Dy1} = \mathbf{0.3}$ kips

Live load in y $F_{Ly1} = \mathbf{0.4}$ kips

Dead load moment in x $M_{Dx1} = \mathbf{2.0}$ kip_ft

Live load moment in x $M_{Lx1} = \mathbf{2.7}$ kip_ft

Dead load moment in y $M_{Dy1} = \mathbf{1.2}$ kip_ft

Live load moment in y $M_{Ly1} = \mathbf{1.7}$ kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.459)

1.0D + 1.0L (0.772)

1.0D + 1.0Lr (0.459)

1.0D + 1.0S (0.459)

1.0D + 1.0R (0.459)

1.0D + 0.75L + 0.75Lr (0.694)

1.0D + 0.75L + 0.75S (0.694)

1.0D + 0.75L + 0.75R (0.694)

1.0D + 0.6W (0.459)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.523)

1.0D + 0.75L + 0.75Lr + 0.45W (0.694)

1.0D + 0.75L + 0.75S + 0.45W (0.694)

1.0D + 0.75L + 0.75R + 0.45W (0.694)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.740)

0.6D + 0.6W (0.275)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.211)

Combination 2 results: 1.0D + 1.0L

Forces on foundation

Force in x-axis $F_{dx} = \gamma_D \times F_{Dx1} + \gamma_L \times F_{Lx1} = \mathbf{0.7}$ kips

Force in y-axis $F_{dy} = \gamma_D \times F_{Dy1} + \gamma_L \times F_{Ly1} = \mathbf{0.7}$ kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{29.0}$ kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h) + \gamma_L \times (F_{Lz1} \times x_1 + M_{Lx1} + F_{Lx1} \times h) = \mathbf{78.1}$ kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1 + M_{Dy1} + F_{Dy1} \times h) + \gamma_L \times (F_{Lz1} \times y_1 + M_{Ly1} + F_{Ly1} \times h) = \mathbf{76.3}$ kip_ft

Uplift verification

Vertical force F_{dz} = **28.964** kips

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Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_D \times (M_{Dx1}+F_{Dx1} \times h) + \gamma_L \times (M_{Lx1}+F_{Lx1} \times h)$ = **5.72** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_L \times (F_{Lz1} \times (x_1 - L_x))$ = **-72.41** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL})$ = **12.659**

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Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_D \times (M_{Dy1}+F_{Dy1} \times h) + \gamma_L \times (M_{Ly1}+F_{Ly1} \times h)$ = **3.92** kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (F_{Dz1} \times (y_1 - L_y)) + \gamma_L \times (F_{Lz1} \times (y_1 - L_y))$ = **-72.41** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL})$ = **18.472**

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Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 kN) \times tan(\delta_{bb})$ = **16.722** kips

Resultant horizontal force $F_{dr} = \sqrt{(F_{dx}^2 + F_{dy}^2)}$ = **0.962** kips

Resultant horizontal force angle $\alpha = Atan(F_{dy} / F_{dx})$ = **45** deg

Passive soil pressure resistance, x direction $F_{RxPass} = 0.5 \times K_P \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil}$ = **5.156** kips

Passive soil pressure resistance, y direction $F_{RyPass} = 0.5 \times K_P \times (h^2 + 2 \times h \times h_{soil}) \times L_x \times \gamma_{soil}$ = **5.156** kips

Resultant soil pressure resistance $F_{RrPass} = abs(F_{RxPass} \times cos(\alpha)) + abs(F_{RyPass} \times sin(\alpha))$ = **7.292** kips

Total sliding resistance $F_{Rr} = F_{RFriction} + F_{RrPass}$ = **24.015** kips

Factor of safety $abs(F_{Rr} / F_{dr})$ = **24.972**

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Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **2.37** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **1.624** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.696** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.072** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.245** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.621** ksf

Minimum base pressure $q_{min} = min(q_1,q_2,q_3,q_4)$ = **0.696** ksf

Maximum base pressure $q_{max} = max(q_1,q_2,q_3,q_4)$ = **1.621** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2.1** ksf

q_{max} / q_{allow} = **0.772**

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FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete	f'_c = **4000** psi
Yield strength of reinforcement	f_y = **60000** psi
Cover to reinforcement	c_{nom} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.046)
1.2D + 1.6L + 0.5Lr (0.092)
1.2D + 1.6L + 0.5S (0.092)
1.2D + 1.6L + 0.5R (0.092)
1.2D + 1.0L + 1.6Lr (0.072)
1.2D + 1.0L + 1.6S (0.072)
1.2D + 1.0L + 1.6R (0.072)
1.2D + 1.6Lr + 0.5W (0.040)
1.2D + 1.6S + 0.5W (0.040)
1.2D + 1.6R + 0.5W (0.040)
1.2D + 1.0L + 0.5Lr + 1.0W (0.072)
1.2D + 1.0L + 0.5S + 1.0W (0.072)
1.2D + 1.0L + 0.5R + 1.0W (0.072)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.079)
0.9D + 1.0W (0.030)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.024)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in x-axis	$F_{ux} = \gamma_D \times F_{Dx1} + \gamma_L \times F_{Lx1}$ = **1.0** kips
Ultimate force in y-axis	$F_{uy} = \gamma_D \times F_{Dy1} + \gamma_L \times F_{Ly1}$ = **1.0** kips
Ultimate force in z-axis	$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **38.7** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0	$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h) + \gamma_L \times (F_{Lz1} \times x_1 + M_{Lx1} + F_{Lx1} \times h)$ = **104.9** kip_ft
Ultimate moment in y-axis, about y is 0	$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1 + M_{Dy1} + F_{Dy1} \times h) + \gamma_L \times (F_{Lz1} \times y_1 + M_{Ly1} + F_{Ly1} \times h)$ = **102.4** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis	$e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **2.53** in
Eccentricity of base reaction in y-axis	$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **1.736** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.888** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.425** ksf

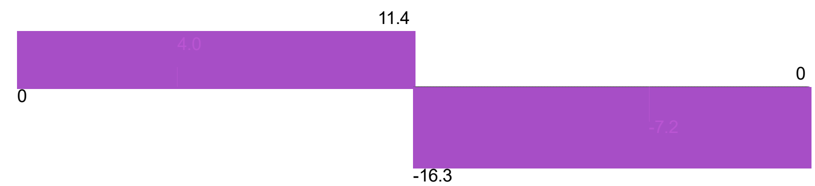
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.671** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.209** ksf

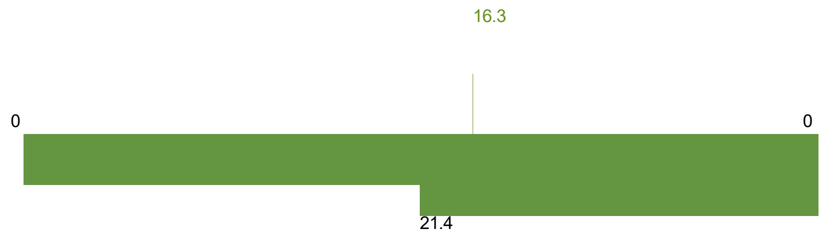
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0.888** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **2.209** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} =$ **16.308** kip_ft

Tension reinforcement provided 6 No.6 bottom bars (10.6 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} =$ **2.64** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_y \times h =$ **1.944** in^2

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Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18\text{ in}) =$ **18** in

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Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2 =$ **14.625** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.776** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.913** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.04503**

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Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **187.925** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **169.133** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.096**

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One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **7.192** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2)$ = **14.625** in

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 11-3) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v$ = **110.996** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **83.247** kips

$V_{u.x} / \phi V_n$ = **0.086**

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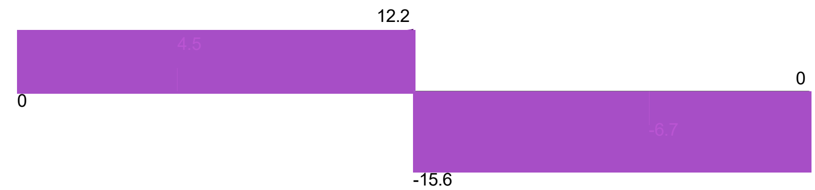
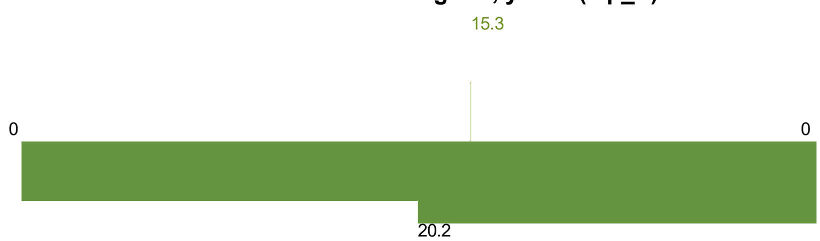
Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **15.283** kip_ft

Tension reinforcement provided 6 No.6 bottom bars (10.6 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **2.64** in^2

Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.944** in^2

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Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18 \text{ in})$ = **18** in

55 5 5 5 5 5 5

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **13.875** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.776** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.913** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04257**

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Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **178.025** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **160.223** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.095**

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One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **6.697** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2)$ = **13.875** in

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 11-3) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **105.304** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **78.978** kips

$V_{u.y} / \phi V_n$ = **0.085**

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Two-way shear design at column 1

Depth to reinforcement d_{v2} = **14.25** in

Shear perimeter length (11.11.1.2) l_{xp} = **22.250** in

Shear perimeter width (11.11.1.2) l_{yp} = **22.250** in

Shear perimeter (11.11.1.2) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **89.000** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **495.063** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **431.063** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **1.747** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **23.193** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **18.287** psi

Column geometry factor (11.11.2.1) $\beta = l_{y1} / l_{x1}$ = **1.00**

Column location factor (11.11.2.1) α_s =**40**

Concrete shear strength (11.11.2.1) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **531.547** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 11-2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (Eq. 11-1) $\phi v_n = \phi_v \times v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.096**

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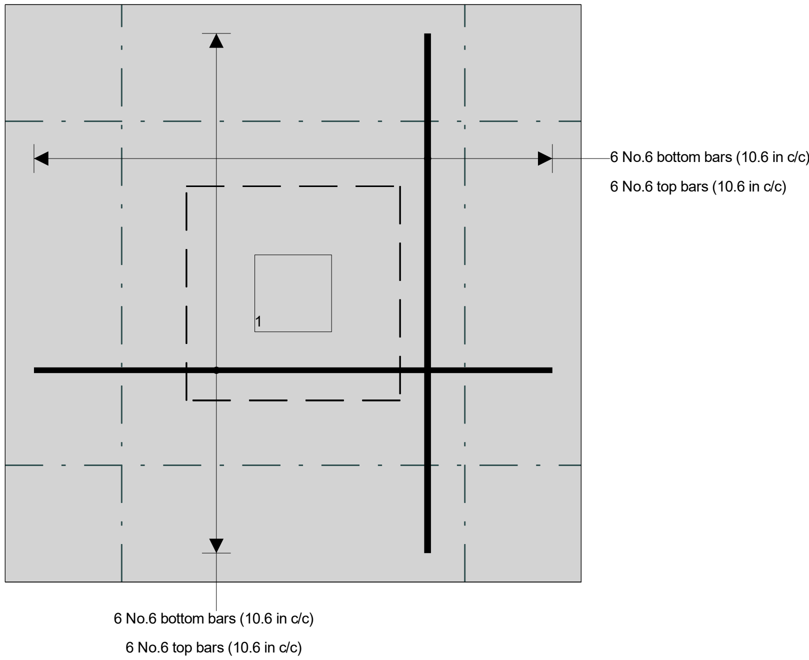


6 No.6 bottom bars (10.6 in c/c)
6 No.6 top bars (10.6 in c/c)
1
6 No.6 bottom bars (10.6 in c/c)
6 No.6 top bars (10.6 in c/c)

RECTANGULAR SPREAD FOOTING ANALYSIS

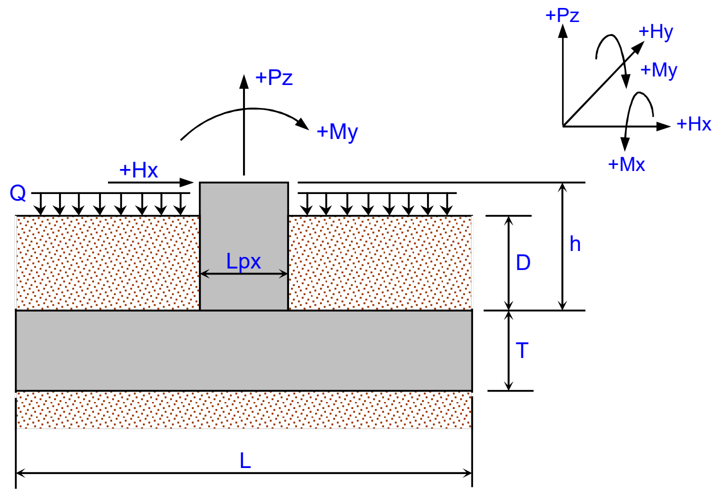
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	I-10 Rest Areas	Subject:	Porch Footer W-6		
Job Number:	B190988.00	Originator:	EJM	Checker:	

Input Data: LC 202

Footing Data:

Footing Length, L =	5.000	ft.
Footing Width, B =	5.000	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.145	kcf
Soil Depth, D =	1.333	ft.
Soil Unit Wt., γs =	0.110	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

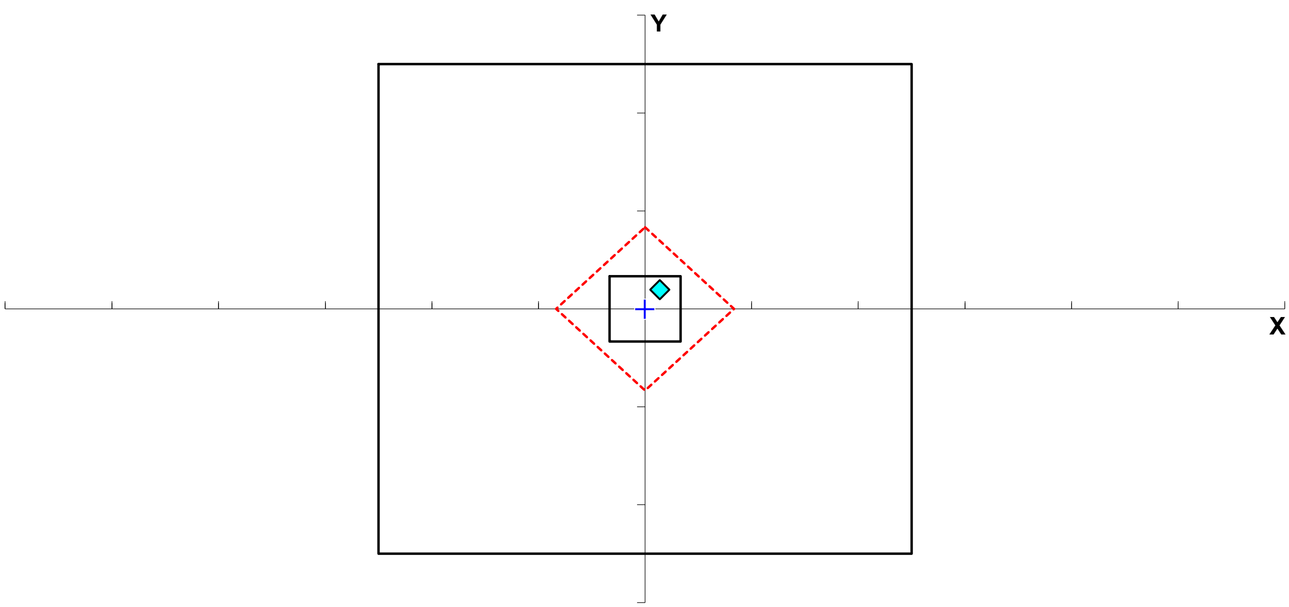


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	0.667							
Lpy (ft.) =	0.667							
h (ft.) =	0.000							
Pz (k) =	-19.861							
Hx (k) =	0.652							
Hy (k) =	0.680							
Mx (ft-k) =	-4.699							
My (ft-k) =	2.895							



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-30.78	kips
ex =	0.14	ft. (<= L/6)
ey =	0.20	ft. (<= B/6)

Overturning Check:

ΣMrx =	76.94	ft-kips
ΣMox =	-6.06	ft-kips
FS(ot)x =	12.699	>= 1.5
ΣMry =	76.94	ft-kips
ΣMoy =	4.20	ft-kips
FS(ot)y =	18.324	>= 1.5

Sliding Check:

Pass(x) =	7.70	kips
Frict(x) =	12.31	kips
FS(slid)x =	30.690	>= 1.5
Pass(y) =	7.70	kips
Frict(y) =	12.31	kips
FS(slid)y =	29.426	>= 1.5

Uplift Check:

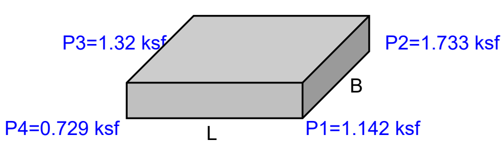
ΣPz(down) =	-30.78	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	5.000	ft.
Brg. Ly =	5.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	6*ex/L + 6*ey/B = 0.408

Gross Soil Bearing Corner Pressures:

P_1 =	1.142	ksf
P_2 =	1.733	ksf
P_3 =	1.320	ksf
P_4 =	0.729	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

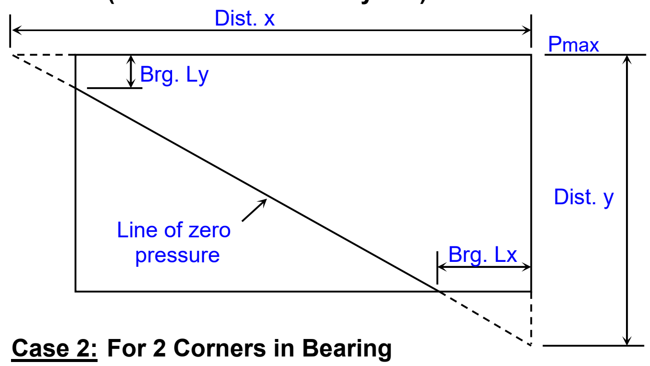
$P_{max}(net) = P_{max}(gross)-(D+T)*\gamma s$

$P_{max}(net)$ = 1.367 ksf

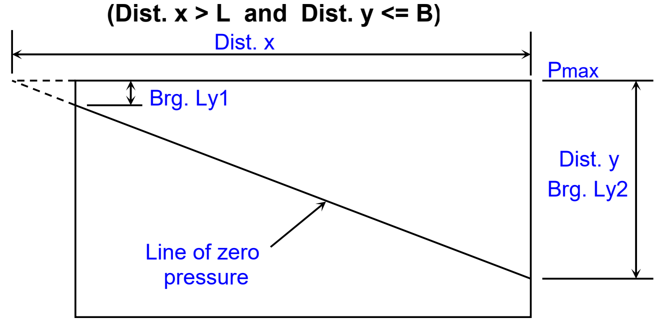
Comments:

Nomenclature for Biaxial Eccentricity:

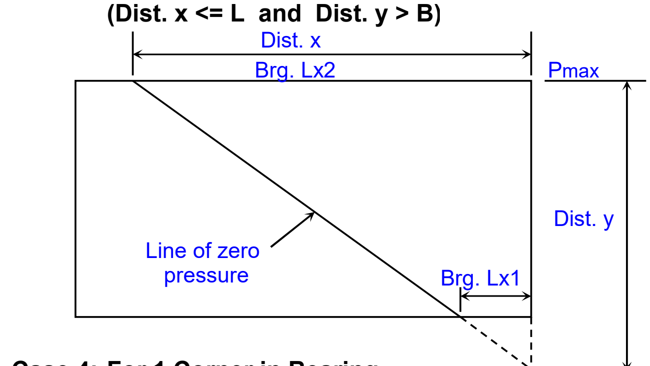
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



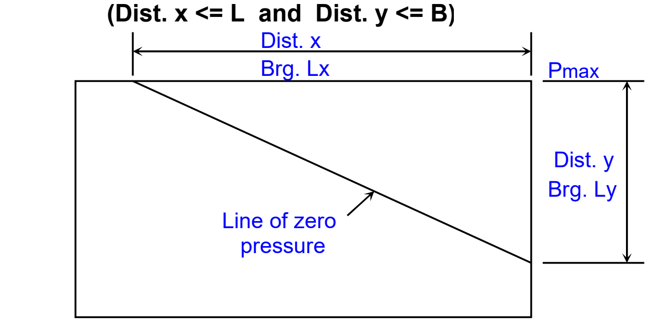
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



STORAGE BUILDING FOUNDATION

References: 1. ACI 318-14
2. ASCE/SEI 7-10

Building Parameters

$W_{CMU} := 61psf$ Weight of CMU wall $W_{Conc} := 145pcf$ Unit weight of Conc.

$W_{EaveHt} := (14ft + 8in)$ Eave height of wall above footing

$W_{MeanHt} := 19ft + 8in$ Mean roof height above footing

$Ftg_t := 12in$ Footing thickness $Ftg_w := 1ft + 6in$ Footing width

$Ext_L := 2ft$ Length of footing extension

$W_L := 30ft$ Shear wall length $Ftg_L := (31ft + 6in)$ Wall footing length

$$I_X := \frac{Ftg_w \cdot Ftg_L^3}{12} + \frac{2 \cdot Ext_L \cdot Ftg_w^3}{12} + 2 \cdot Ext_L \cdot Ftg_w \cdot \left(\frac{Ftg_L}{2} - \frac{Ftg_w}{2}\right)^2 = 5258\,ft^4$$ Combined footing I_X

$$S_X := \frac{2 \cdot I_X}{Ftg_L} = 333.848\,ft^3$$ Combined footing section modulus

$\gamma_{soil} := 110pcf$ Soil density

$F_{allownet} := 2100psf$ Allowable net pressure

$F_{allowgross} := F_{allownet} + 2.333ft \cdot \gamma_{soil} = 2357\,psf$

Loads

Shear wall is not a load bearing wall. Footer extensions are under load bearing walls

$V_H := 23.2kip$ Wind shear to shear wall

$W_{nonbearing} := W_{MeanHt} \cdot W_{CMU} + W_{Conc} \cdot Ftg_w \cdot Ftg_t = 1417\,\frac{lb}{ft}$ Dead load

$W_{bearing} := W_{EaveHt} \cdot W_{CMU} + W_{Conc} \cdot Ftg_w \cdot Ftg_t = 1112\,\frac{lb}{ft}$ Dead load

$R_{DL} := 358\,\frac{lb}{ft}$ Roof dead load

$R_{LLr} := 575\,\frac{lb}{ft}$ Roof live load

$R_{WL} := 259\,\frac{lb}{ft}$ Roof wind load

$M_{WL} := V_H \cdot (W_{EaveHt} + Ftg_t) = 363.5\,kip \cdot ft$

Loading Combinations

3. DL+LLr

6a. DL+0.75LLr+0.45WL

7. 0.6DL+0.6WL

Soil Pressure Check

$F_{allow} := 2100psf + 2.333ft \cdot \gamma_{soil} = 2357\,psf$

For LC 3:

$P_{tot3} := (W_{bearing} + R_{DL} + R_{LLr})Ext_L + W_{nonbearing} \cdot Ftg_w = 6216\,lbf$

$A_{bearing} := Ext_L \cdot Ftg_w + Ftg_w^{\,2} = 5.25\,ft^{2.00}$

$F_{max3} := \frac{P_{tot3}}{A_{bearing}} = 1184\,psf$ < $F_{allow} = 2357psf$ **OK**

For LC 6

$$P_{tot6a} := \left(W_{bearing} + R_{DL} + 0.75 \cdot R_{LLr} + 0.45 \cdot R_{WL}\right) \cdot Ext_L + W_{nonbearing} \cdot Ftg_w = 6162\ \text{lbf}$$

$$F_{vert6a} := \frac{P_{tot6a}}{A_{bearing}} = 1174\ \text{psf}$$

$$F_{mom6a} := \frac{0.45 \cdot M_{WL}}{S_X} = 490\ \text{psf}$$

$$F_{max6a} := F_{vert6a} + F_{mom6a} = 1664\ \text{psf}$$

< F.allow =2357psf **OK**

For LC 7:

$$P_{tot7} := 0.6\left(W_{bearing} + R_{DL} + R_{WL}\right) \cdot Ext_L + 0.6W_{nonbearing} \cdot Ftg_w = 3350\ \text{lbf}$$

$$F_{vert7} := \frac{P_{tot7}}{A_{bearing}} = 638\ \text{psf}$$

$$F_{mom7} := \frac{0.6 \cdot M_{WL}}{S_X} = 653\ \text{psf}$$

$$F_{max7} := F_{vert7} + F_{mom7} = 1291\ \text{psf}$$

< F.allow =2357psf **OK**

ATTACHMENT C
Storage Building Design

Storage Building Roof Connections

Roof Loads

$Live_{roof}$ = 20psf
$Dead_{roof}$ = 20psf

Wind loads

Wind Load Case A

$p_{A2} := q_z \cdot (GC_{pA2} - GC_{pin}) = \mathbf{10.387} \cdot psf$

$p_{A2E} := q_z \cdot (GC_{pA2E} - GC_{pin}) = \mathbf{11.985} \cdot psf$

$p_{A2.1} := q_z \cdot (GC_{pA2} - GC_{pip}) = \mathbf{0.799} \cdot psf$

$p_{A2.1E} := q_z \cdot (GC_{pA2E} - GC_{pip}) = \mathbf{2.397} \cdot psf$

$p_{A3} := q_z \cdot (GC_{pA3} - GC_{pin}) = \mathbf{-6.659} \cdot psf$

$p_{A3E} := q_z \cdot (GC_{pA3E} - GC_{pin}) = \mathbf{-9.322} \cdot psf$

$p_{A3.1} := q_z \cdot (GC_{pA3} - GC_{pip}) = \mathbf{-16.247} \cdot psf$

$p_{A3.1E} := q_z \cdot (GC_{pA3E} - GC_{pip}) = \mathbf{-18.91} \cdot psf$

Wind Load Case B

$p_{B2} := q_z \cdot (GC_{pB2} - GC_{pin}) = \mathbf{-13.583} \cdot psf$

$p_{B2E} := q_z \cdot (GC_{pB2E} - GC_{pin}) = \mathbf{-23.704} \cdot psf$

$p_{B2.1} := q_z \cdot (GC_{pB2} - GC_{pip}) = \mathbf{-23.172} \cdot psf$

$p_{B2.1E} := q_z \cdot (GC_{pB2E} - GC_{pip}) = \mathbf{-33.293} \cdot psf$

$p_{B3} := q_z \cdot (GC_{pB3} - GC_{pin}) = \mathbf{-5.061} \cdot psf$

$p_{B3E} := q_z \cdot (GC_{pB3E} - GC_{pin}) = \mathbf{-9.322} \cdot psf$

$p_{B3.1} := q_z \cdot (GC_{pB3} - GC_{pip}) = \mathbf{-14.649} \cdot psf$

$p_{B3.1E} := q_z \cdot (GC_{pB3E} - GC_{pip}) = \mathbf{-18.91} \cdot psf$

Envelope wind loads:
$Wind_{down}$: 12psf
$Wind_{uplift}$: -33.3psf

Roof Envelope Load:
max down: $Dead_{roof}$ + .75 $Live_{roof}$ + .75 (.6 $Wind_{down}$) = 40.4psf
max uplift: .6 $Dead_{roof}$ + .6 $Wind_{uplift}$ = -7.8psf

Conservative worst case loading

$a := 2.5ft \quad l := 1.5ft$

$w := 40.4psf \cdot 2ft = 80.8\,plf$

$R_1 := \frac{w}{2 \cdot l} \cdot (l^2 - a^2) = -107.7333\,lbf \qquad R_1 := \frac{w \cdot a^2}{2 \cdot l} = 168.333\,lbf$

$R_2 := \frac{w}{2 \cdot l} \cdot (l + a)^2 = 430.9333\,lbf$

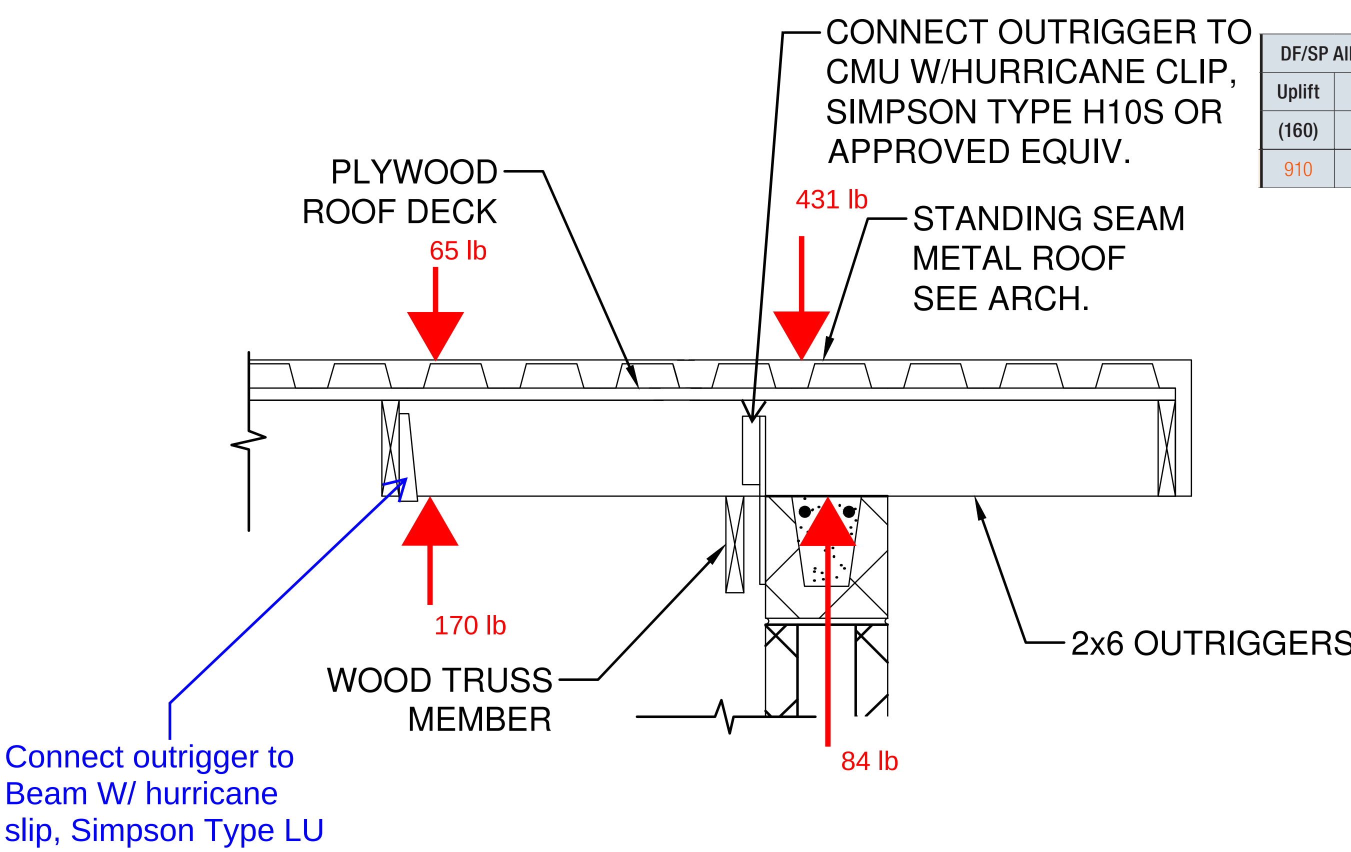
$w := -7.8psf \cdot 2ft = -15.6\,plf$

$R_1 := \frac{w}{2 \cdot l} \cdot (l^2 - a^2) = 20.8\,lbf$

$R_2 := \frac{w}{2 \cdot l} \cdot (l + a)^2 = -83.2\,lbf$

$a := 1.5ft \quad l := 2.5ft$

$R_{.1} := \frac{w}{2 \cdot l} \cdot (l^2 - a^2) = 64.64\,lbf$



DF/SP Allowable Loads		
Uplift	Lateral (160)	
(160)	F_1	F_2
910	660	215

DF/SP Allowable Loads			
Uplift (160)	Floor (100)	Snow (115)	Roof (125)
540	835	950	1,030

H/TSP

Beam (2X6) to CMU Wall Connection
Max Downward Load: 431 lb
Max Uplift: 84 lb

Seismic and Hurricane Ties

Simpson Strong-Tie hurricane ties provide a positive connection between truss/rafter and the wall of the structure to resist wind and seismic forces.

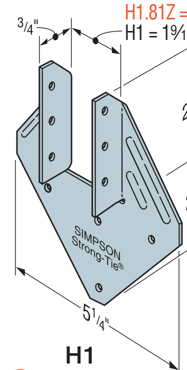
Material: See table

Finish: Galvanized. H1.81Z, H7Z and H11Z — ZMAX® coating. Some models available in stainless steel or ZMAX; see Corrosion Information, pp. 13–15 or visit **strongtie.com**.

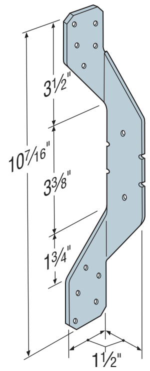
Installation:

- Use all specified fasteners; see General Notes.
- Hurricane ties can be installed with flanges facing inward or outward.
- H2.5T, H3 and H6 ties are shipped in equal quantities of right and left versions (right versions shown).
- Hurricane ties do not replace solid blocking.
- When installing ties on plated trusses (on the side opposite the truss plate) do not fasten through the truss plate from behind. This can force the truss plate off of the truss and compromise truss performance.
- H10A optional nailing to connect shear blocking, use 8d nails. Slots allow maximum field bending up to a pitch of 6:12, use H10A sloped loads for field-bent installation.

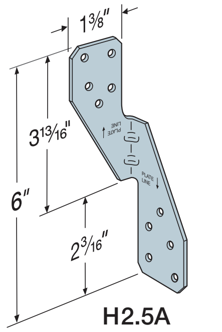
Codes: See p. 12 for Code Reference Key Chart



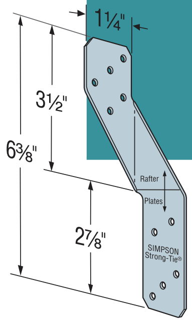
H1
NEW (H1.81Z similar)



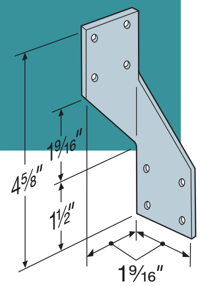
H2A
(H2ASS similar)



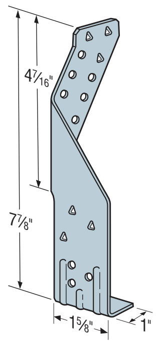
H2.5A
(H2.5ASS similar)



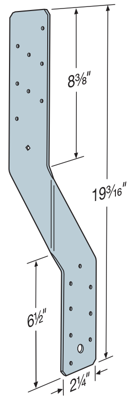
H2.5T



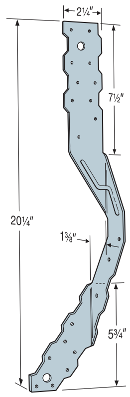
H3



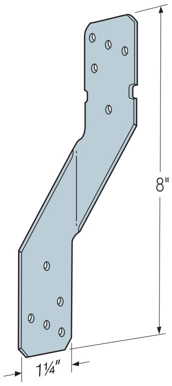
TSP



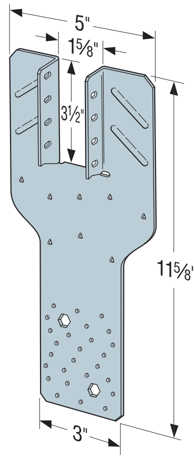
H6



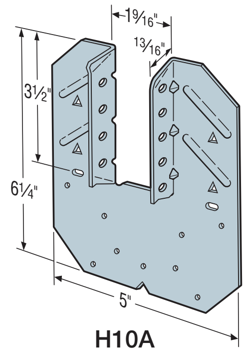
H7Z



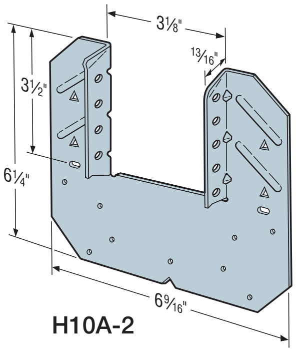
H8



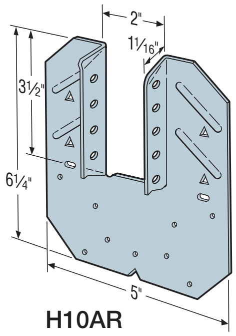
H10S



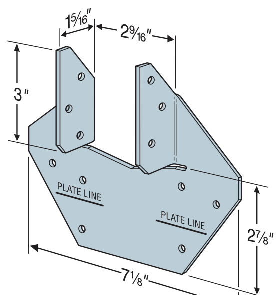
H10A
(H10ASS similar)



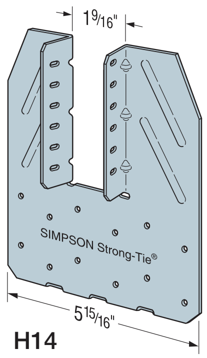
H10A-2



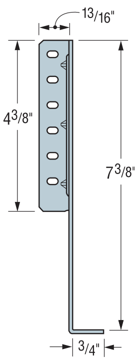
H10AR



H11Z



H14



H14 Profile

Straps and Ties

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Model No.	Ga.	Fasteners (in.)			DF/SP Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	Code Ref.
		To Rafters/ Truss	To Plates	To Studs	Uplift (160)	Lateral (160) F1	Lateral (160) F2		Uplift (160)	Lateral (160) F1	Lateral (160) F2		
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	350	335	195	330	300	290	150	260	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	565	110	110	575	535	110	110	495	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	495	135	145	420	495	135	145	420	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	415	365	180	145	290	
H6	16	—	(8) 0.131 x 2½	(8) 0.131 x 2½	1,230	—	—	—	1,055	—	—	—	IBC, FL
H7Z	16	(4) 0.131 x 2½	(2) 0.131 x 1½	(8) 0.131 x 2½	830	410	—	—	715	355	—	—	
H8	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	780	95	90	630	710	95	90	510	IBC, FL, LA
H10A Field Bent	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	855	590	285	—	760	505	285	—	
H10A	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,040	565	285	—	1,015	485	285	—	
H10ASS	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	970	565	170	—	835	485	170	—	—
H10AR	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,050	490	285	—	905	420	285	—	
H10S	18	(8) 0.131 x 1½	(8) 0.131 x 1½	(8) 0.131 x 2½	910	660	215	550	785	570	185	475	IBC, FL, LA
H10A-2	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,080	680	260	—	930	585	225	—	
H11Z	18	(6) 0.162 x 2½	(6) 0.162 x 2½	—	830	525	760	—	715	450	655	—	—
H14	18	(12) 0.131 x 1½	(13) 0.131 x 2½	—	1,275	725	285	—	1,050	480	245	—	IBC, FL, LA
		(12) 0.131 x 1½	(15) 0.131 x 2½	—	1,340	670	230	—	1,050	480	245	—	
TSP	16	(9) 0.148 x 1½	(6) 0.148 x 1½	—	755	310	190	—	650	265	160	—	FL
		(9) 0.148 x 1½	(6) 0.148 x 3	—	1,015	310	190	—	875	265	160	—	

1. See pp. 260–261 for Straps and Ties General Notes.
2. Allowable loads are for one anchor. A minimum rafter thickness of 2½" must be used when framing anchors are used on each side of the joist and on the same side of the plate (exception: connectors installed such that nails on opposite side don't interfere).
3. Allowable DF/SP uplift load for stud-to-bottom plate installation (see detail 15) is 390 lb. (H2.5A); 265 lb. (H2.5ASS); and 310 lb. (H8). For SPF/HF values, multiply these values by 0.86.
4. Allowable loads in the F_1 direction are not intended to replace diaphragm boundary members or cross-grain bending of the truss or rafter members.
5. When cross-grain bending or cross-grain tension cannot be avoided in the members, mechanical reinforcement to resist such forces shall be considered by the Designer.
6. Hurricane ties are shown on the outside of the wall for clarity and assume a minimum overhang of 3½". Installation on the inside of the wall is acceptable (see General Instructions for the Installer, note "s" on p. 18). For uplift continuous load path, install connectors in the same area (e.g., truss-to-plate connector and plate-to-stud connector) on the same side of the wall, unless detailed by designer. See technical bulletin T-C-HTIECON at **strongtie.com** for more information.
7. Southern pine allowable uplift loads for H10A = 1,340 lb. and for the H14 = 1,465 lb.
8. Refer to Simpson Strong-Tie® technical bulletin T-C-HTIEBEAR at **strongtie.com** for allowable bearing enhancement loads.
9. H10S can have the stud offset a maximum of 1" from the rafter (center to center) for a reduced uplift of 890 lb. (DF/SP) and 765 lb. (SPF).
10. H10S nails to plates are optional for uplift but required for lateral loads.
11. Some load values for the stainless-steel connectors shown here are lower than those for the carbon-steel versions. Ongoing test programs have shown this also to be the case with other stainless-steel connectors in the product line that are installed with nails. Visit **strongtie.com/corrosion** for updated information.
12. The allowable loads of stainless-steel connectors match carbon-steel connectors when installed with stainless-steel Strong-Drive® SCNR Ring-Shank Connector nails. For more information, refer to engineering letter L-F-SSNAILS at **strongtie.com**.
13. Allowable DF/SP/SPF uplift load for the H2.25A fastened to a 2x4 truss bottom chord and double top plates using (5) 0.131" x 1½" nails in the top plates and (3) 0.131" x 1½" nails in the lowest three flange holes into the truss bottom chord is 260 lb. (160).
14. For TSP installed stud to single plate see p. 276.
15. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

Beam (2X6) to Beam (2X6) Connection
Max Downward Load: 65lb
Max Uplift: 170 lb

Double-Shear Face-Mount Joist Hangers (cont.)

HHUS/HGUS

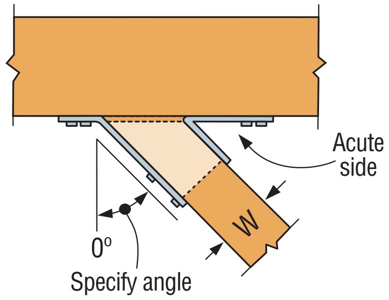
HHUS — Sloped and/or Skewed Seat

- HHUS hangers can be skewed to a maximum of 45° and/or sloped to a maximum of 45°
- For skew only, maximum allowable download is 0.85 of the table load
- For sloped only or sloped and skewed hangers, the maximum allowable download is 0.65 of the table load
- Uplift loads for sloped/skewed conditions are 0.72 of the table load, not to exceed 2,475 lb.
- The joist must be bevel-cut to allow for double-shear nailing

HGUS — Skewed Seat

- HGUS hangers can be skewed only to a maximum of 45°. Allowable loads are:

HGUS Seat Width	Joist	Download	Uplift
W < 2"	Square cut	0.62 of table load	0.46 of table load
W < 2"	Bevel cut	0.72 of table load	0.46 of table load
2" < W < 6"	Bevel cut	0.85 of table load	0.41 of table load
2" < W < 6"	Square cut	0.46 of table load	0.41 of table load
W > 6"	Bevel cut	0.85 of table load	0.41 of table load



Top View HHUS Hanger Skewed Right
(joist must be bevel cut)
All joist nails installed on the outside angle *(non-acute side).*

HUCQ

Heavy-Duty Face-Mount Joist Hanger

The HUCQ series are heavy-duty joist hangers that incorporate Strong-Drive® SDS Heavy-Duty Connector screws. Designed and tested for installation at the end of a beam or on a post, they provide a strong connection with fewer fasteners than nailed hangers. See pp. 144–150 for structural composite lumber hangers.

Material: 14 gauge

Finish: Galvanized. Most models available in stainless steel or ZMAX® coating.

Installation:

- Use all specified fasteners; see General Notes.
- Install ¼" x 2½" Strong-Drive SDS Heavy-Duty Connector screws, which are provided, in all round holes. (Lag screws will not achieve the same load.)
- HUCQ hangers can be welded to a steel member. Allowable loads are the lesser of the values in the hanger tables on pp. 104–113 or the weld capacity — refer to technical bulletin T-C-HUHUC-W at **strongtie.com**.

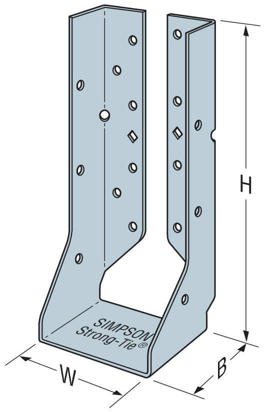
Allowable Loads:

- See table on pp. 104–113 for loads.

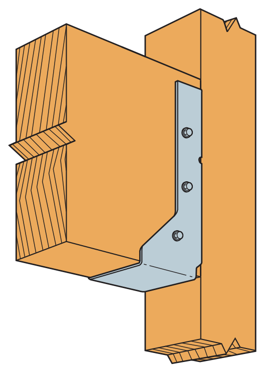
Options:

- These hangers cannot be modified.

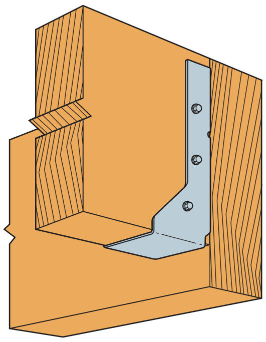
Codes: See p. 12 for Code Reference Key Chart



HUCQ410



Typical HUCQ Installation on a Post



Typical HUCQ Installation on a Beam

SIMPSON Strong-Tie®

Face-Mount Hangers – Solid Sawn Lumber (DF/SP)

The Joist Hanger Selector software enables you the most optimum product for your project.
The software takes into consideration all the characteristics seen in this catalog. Visit **strongtie.com/jhs**.

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

	Joist Size	Model No.	Ga.	Dimensions (in.) W	H	B	Min./Max.	Fasteners (in.) Header	Joist	DF/SP Allowable Loads Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Installed Cost Index (ICI)	Code Ref.
		Sawn Lumber Sizes													
	2X4	LU24	20	1 9/16	3⅛	1½	—	(4) 0.162 x 3½	(2) 0.148 x 1½	240	555	630	655	Lowest	IBC, FL, LA
	2X4	LUS24	18	1 9/16	3⅛	1¾	—	(4) 0.148 x 3	(2) 0.148 x 3	435	670	765	820	3%	IBC, FL, LA
	2X4	U24	16	1 9/16	3⅛	1½	—	(4) 0.162 x 3½	(2) 0.148 x 1½	240	575	650	705	67%	IBC, FL, LA
	2X4	HU26	14	1 9/16	3 1/16	2¼	—	(4) 0.162 x 3½	(2) 0.148 x 1½	305	595	670	720	295%	IBC, FL, LA
	DBL 2X4	LUS24-2	18	3⅛	3⅛	2	—	(4) 0.162 x 3½	(2) 0.162 x 3½	410	800	905	980	Lowest	IBC, FL, LA
	DBL 2X4	U24-2	16	3⅛	3	2	—	(4) 0.162 x 3½	(2) 0.148 x 3	240	575	650	705	33%	IBC, FL, LA
	DBL 2X4	HU24-2 / HUC24-2	14	3⅛	3 1/16	2½	—	(4) 0.162 x 3½	(2) 0.148 x 3	380	595	670	720	240%	IBC, FL, LA
SS	2x6	LUS26	18	1 9/16	4¾	1¾	—	(4) 0.148 x 3	(4) 0.148 x 3	1,165	865	990	1,060	Lowest	IBC, FL, LA
	2x6	LU26	20	1 9/16	4¾	1½	—	(6) 0.162 x 3½	(4) 0.148 x 1½	540	835	950	1,030	6%	IBC, FL, LA
	2x6	U26	16	1 9/16	4¾	2	—	(6) 0.162 x 3½	(4) 0.148 x 1½	535	865	980	1,055	43%	IBC, FL, LA
SS	2x6	LUC26Z	18	1 9/16	4¾	1¾	—	(6) 0.162 x 3½	(4) 0.148 x 1½	730	845	965	1,040	160%	IBC, FL, LA
	2x6	HU26	14	1 9/16	3 1/16	2¼	—	(4) 0.162 x 3½	(2) 0.148 x 1½	305	595	670	720	179%	IBC, FL, LA
	2x6	HUS26	16	1⅝	5⅜	3	—	(14) 0.162 x 3½	(6) 0.162 x 3½	1,320	2,735	3,095	3,235	276%	IBC, FL, LA
SS	DBL 2X6	LUS26-2	18	3⅛	4⅞	2	—	(4) 0.162 x 3½	(4) 0.162 x 3½	1,060	1,030	1,170	1,265	Lowest	IBC, FL, LA
	DBL 2X6	U26-2	16	3⅛	5	2	—	(8) 0.162 x 3½	(4) 0.148 x 3	535	1,150	1,305	1,410	65%	IBC, FL, LA
	DBL 2X6	HUS26-2	14	3⅛	5 3/16	2	—	(4) 0.162 x 3½	(4) 0.162 x 3½	1,165	1,055	1,195	1,290	172%	IBC, FL, LA
	DBL 2X6	HU26-2 / HUC26-2	14	3⅛	4 15/16	2½	Min.	(8) 0.162 x 3½	(4) 0.148 x 3	755	1,190	1,345	1,440	233%	IBC, FL, LA
	DBL 2X6	HU26-2 / HUC26-2	14	3⅛	4 15/16	2½	Max.	(12) 0.162 x 3½	(6) 0.148 x 3	1,135	1,785	2,015	2,165	254%	IBC, FL, LA
	TPL 2x6	LUS26-3	18	4⅝	4⅛	2		(4) 0.162 x 3½	(4) 0.162 x 3½	1,060	1,030	1,170	1,265	*	IBC, FL
	TPL 2x6	U26-3	16	4⅝	4¼	2		(8) 0.162 x 3½	(4) 0.148 x 3	535	1,150	1,305	1,410	*	IBC, FL
	TPL 2x6	HU26-3 / HUC26-3	14	4 11/16	4 19/32	2½	Min.	(8) 0.162 x 3½	(4) 0.148 x 3	755	1,190	1,345	1,440	*	IBC, FL
	TPL 2x6	HU26-3 / HUC26-3	14	4 11/16	4 19/32	2½	Max.	(12) 0.162 x 3½	(6) 0.148 x 3	1,135	1,785	2,015	2,165	*	IBC, FL
SS	2x8	LUS26	18	1 9/16	4¾	1¾	—	(4) 0.148 x 3	(4) 0.148 x 3	1,165	865	990	1,060	Lowest	IBC, FL, LA
	2x8	LU26	20	1 9/16	4¾	1½	—	(6) 0.162 x 3½	(4) 0.148 x 1½	540	835	950	1,030	6%	IBC, FL, LA
SS	2x8	LUS28	18	1 9/16	6⅝	1¾	—	(6) 0.148 x 3	(4) 0.148 x 3	1,165	1,100	1,260	1,350	23%	IBC, FL, LA
	2x8	LU28	20	1 9/16	6⅝	1½	—	(8) 0.162 x 3½	(6) 0.148 x 1½	850	1,110	1,180	1,180	39%	IBC, FL, LA
	2x8	U26	16	1 9/16	4¾	2	—	(6) 0.162 x 3½	(4) 0.148 x 1½	535	865	980	1,055	43%	IBC, FL, LA
SS	2x8	LUC26Z	18	1 9/16	4¾	1¾	—	(6) 0.162 x 3½	(4) 0.148 x 1½	730	845	965	1,040	160%	IBC, FL, LA
	2x8	HU28	14	1 9/16	5¼	2¼	—	(6) 0.162 x 3½	(4) 0.148 x 1½	605	895	1,010	1,080	251%	IBC, FL, LA
	2x8	HUS26	16	1⅝	5⅜	3	—	(14) 0.162 x 3½	(6) 0.162 x 3½	1,320	2,735	2,845	2,845	276%	IBC, FL, LA
	2x8	HUS28	16	1⅝	7	3	—	(22) 0.162 x 3½	(8) 0.162 x 3½	1,760	4,095	4,095	4,095	409%	IBC, FL, LA
SS	DBL 2x8	LUS26-2	18	3⅛	4⅞	2	—	(4) 0.162 x 3½	(4) 0.162 x 3½	1,060	1,030	1,170	1,265	Lowest	IBC, FL, LA
SS	DBL 2x8	LUS28-2	18	3⅛	7	2	—	(6) 0.162 x 3½	(4) 0.162 x 3½	1,060	1,315	1,490	1,610	8%	IBC, FL, LA
	DBL 2x8	U26-2	16	3⅛	5	2	—	(8) 0.162 x 3½	(4) 0.148 x 3	535	1,150	1,305	1,410	65%	IBC, FL, LA
	DBL 2x8	HUS28-2	14	3⅛	7 3/16	2	—	(6) 0.162 x 3½	(6) 0.162 x 3½	1,320	1,580	1,790	1,930	188%	IBC, FL, LA
SS	DBL 2x8	HU28-2 / HUC28-2	14	3⅛	6 5/16	2½	Min.	(10) 0.162 x 3½	(4) 0.148 x 3	755	1,490	1,680	1,800	397%	IBC, FL, LA
SS	DBL 2x8	HU28-2 / HUC28-2	14	3⅛	6 5/16	2½	Max.	(14) 0.162 x 3½	(6) 0.148 x 3	1,135	2,085	2,350	2,530	418%	IBC, FL, LA
	TPL 2X8	LUS28-3	18	4⅝	6¼	2	—	(6) 0.162 x 3½	(4) 0.162 x 3½	1,060	1,315	1,490	1,610	*	IBC, FL
	TPL 2X8	U26-3	16	4⅝	4¼	2	—	(8) 0.162 x 3½	(4) 0.148 x 3	535	1,150	1,305	1,410	*	IBC, FL
	TPL 2X8	HU26-3 / HUC26-3	14	4 11/16	4⅝	2½	Min.	(8) 0.162 x 3½	(4) 0.148 x 3	755	1,190	1,345	1,440	*	IBC, FL
	TPL 2X8	HU26-3 / HUC26-3	14	4 11/16	4⅝	2½	Max.	(12) 0.162 x 3½	(6) 0.148 x 3	1,135	1,785	2,015	2,165	*	IBC, FL
	QUAD 2X8	HU28-4 / HUC28-4	14	6⅛	7	2½	Min.	(10) 0.162 x 3½	(4) 0.162 x 3½	755	1,490	1,680	1,800	*	—
	QUAD 2X8	HU28-4 / HUC28-4	14	6⅛	7	2½	Max.	(14) 0.162 x 3½	(6) 0.162 x 3½	1,135	2,085	2,350	2,530	*	—

See footnotes on p. 108.

Codes: See p. 12 for Code Reference Key Chart

UPDATED 06/01/19

Storage Building Foundation

STORAGE BUILDING FOUNDATION

References: 1. ACI 318-14

Building Parameters

$L_{bldg} := 20ft$

$B_{bldg} := 18ft$

$H_{eave} := 10ft + 4in$

$H_{mean} := 13ft + 4in$

$t_{wall} := 8in$ $t_{slab} := 6in$

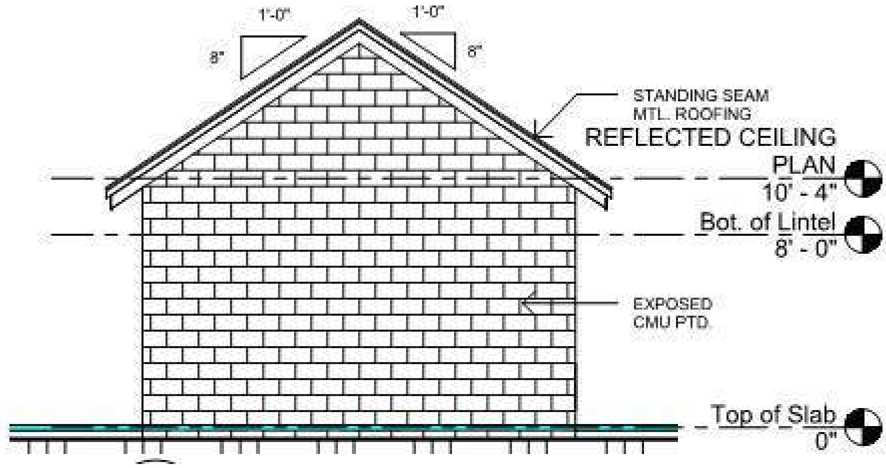


Figure 1

Building makeup is assumed to be 1 1/2" metal deck supporting 2" rigid insulation and a standing seam cover. The roof is supported by wood trusses on 2' centers that span to CMU bearing walls

Walls are supported on continuous spread footers

Floor will be a 6" slab on grade

For floor loading assume storage areas of 250 psf with a 2 ton rated fork lift

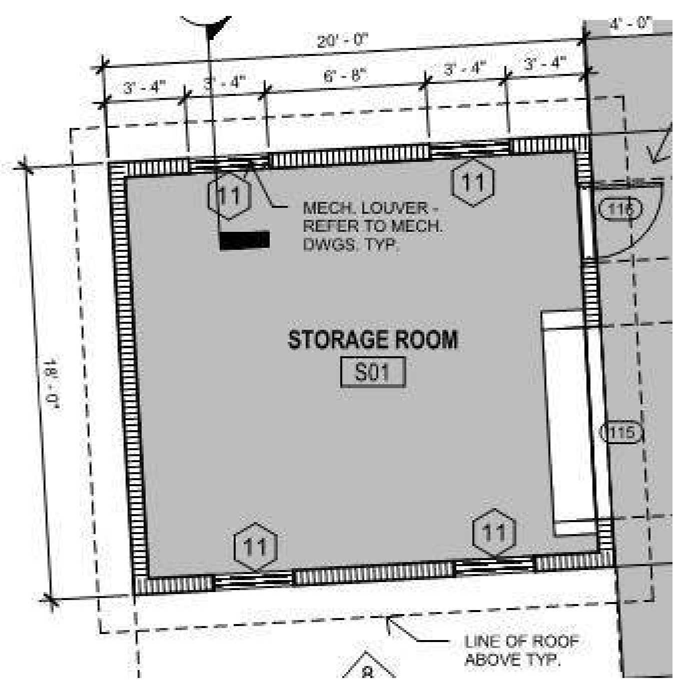


Figure 2

Load Generation

$DL_{roof} := 20psf$

This will include the roof and supporting trusses

$LL_{roof} := 20psf$ Minimum

$Roof_{ovhg} := 1ft + 8in$ $Wt_{CMU} := 61psf$ Assiming reinforcement at 24" on center

Side Wall Loads

$$W_{DLroof} := (L_{bldg} + Roof_{ovhg}) \cdot (B_{bldg} + Roof_{ovhg}) \cdot DL_{roof} = 8522.222\,lbf$$

Total roof DL

$$W_{DLwall} := \frac{W_{DLroof}}{2 \cdot L_{bldg}} = 213.056 \cdot \frac{lb}{ft}$$

Applied wall dead load from roof

$$W_{LLroof} := (L_{bldg} + Roof_{ovhg}) \cdot (B_{bldg} + Roof_{ovhg}) \cdot LL_{roof} = 8522.222\,lbf$$

Total roof LL

$$W_{LLwall} := \frac{W_{LLroof}}{2 \cdot L_{bldg}} = 213.056 \cdot \frac{lb}{ft}$$

Applied wall live load from roof

$$Wt_{brgwall} := H_{eave} \cdot Wt_{CMU} = 630.333 \cdot \frac{lb}{ft}$$

End Wall Loads

CMU wall extends to roof line

$$\gamma_{conc} := 150pcf$$

Assume a 1'-0" concrete raked bond beam along the top

$$A_{\Delta} := \frac{B_{bldg} \cdot 6ft}{2} = 54\,ft^2$$

Triangular wall area above eave

$$A_{CMU} := \frac{(B_{bldg} - 2ft) \cdot 5ft}{2} = 40\,ft^2$$

Triangular CMU wall area above eave

$$Wt_{Addcmu} := A_{CMU} \cdot Wt_{CMU} = 2440\,lbf$$

$$Wt_{Addconc} := (A_{\Delta} - A_{CMU}) \cdot \gamma_{conc} \cdot t_{wall} = 1400 \cdot lb$$

$$Wt_{Add} := Wt_{Addcmu} + Wt_{Addconc} = 3840 \cdot lb$$

$$Wt_{endwall} := \frac{Wt_{Add}}{B_{bldg}} = 213.333 \cdot \frac{lb}{ft}$$

Footing Loads

Side Wall

$$W_{asd} := W_{DLwall} + Wt_{brgwall} + W_{LLwall} = 1056.444 \cdot \frac{lb}{ft}$$ ASD DL+LL to footing

End Wall

$$W_{asdend} := Wt_{endwall} + Wt_{brgwall} = 843.667 \cdot \frac{lb}{ft}$$ ASD DL+LL to footing

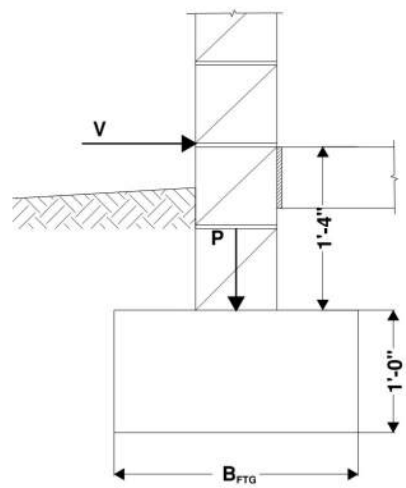


Figure 3

Additional DL below grade

$H_{below} := 1ft + 4in$ $\gamma_{soil} := 110pcf$

$$CMU := H_{below} \cdot Wt_{CMU} = 81.333 \cdot \frac{lb}{ft}$$

Assume 2'-0" wide footer $B_{ftr} := 2ft$

$$SOIL := (B_{ftr} - t_{wall}) \cdot (H_{below} - t_{slab}) \cdot \gamma_{soil} = 122.222 \cdot \frac{lb}{ft}$$

$$CONC := \frac{(B_{ftr} - t_{wall})}{2} \cdot t_{slab} \cdot \gamma_{conc} = 50 \cdot \frac{lb}{ft}$$

For Wind Load:

Assume wall is pinned at the top and bottom

Roof diaphragm takes the top reaction to shear walls

$WL_{wall} := 19.709psf$ Max from the wind load analysis

$$R_{flr} := \frac{WL_{wall} \cdot H_{eave}}{2} = 101.830 \cdot \frac{lb}{ft}$$ Shear per foot of wall

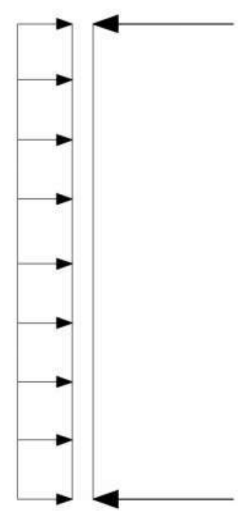


Figure 4

Load Combinations (Side Wall)

LC3 DL+LL_r

LC 5 DL+0.6WL

LC6a DL+0.75LL_r+0.45WL

LC7 0.6DL+0.6WL

$$P_{DL} := W_{DLwall} + Wt_{brgwall} + CMU + SOIL + CONC = 1096.944 \cdot \frac{lb}{ft}$$

$$P_{LL} := W_{LLwall} = 213.056 \cdot \frac{lb}{ft}$$

$$P_{WLn} := -433 \frac{lb}{ft}$$

$$P_{WLp} := 111 \frac{lb}{ft}$$

$$P_{LC3} := P_{DL} + P_{LL} = 1310 \cdot \frac{lb}{ft}$$

$$P_{LC5n} := P_{DL} + 0.6 \cdot P_{WLn} = 837.144 \cdot \frac{lb}{ft}$$

$$P_{LC5p} := P_{DL} + 0.6P_{WLp} = 1163.544 \cdot \frac{lb}{ft}$$

$$P_{LC6an} := P_{DL} + 0.75 \cdot P_{LL} + 0.45 \cdot P_{WLn} = 1061.886 \cdot \frac{lb}{ft}$$

$$P_{LC6ap} := P_{DL} + 0.75 \cdot P_{LL} + 0.45P_{WLp} = 1306.686 \cdot \frac{lb}{ft}$$

$$P_{LC7n} := 0.6 \cdot P_{DL} + 0.6 \cdot P_{WLn} = 398.367 \cdot \frac{lb}{ft}$$

$$P_{LC7p} := 0.6 \cdot P_{DL} + 0.6 \cdot P_{WLp} = 724.767 \cdot \frac{lb}{ft}$$

By inspection LC3 will govern for maximum pressure and LC7n will govern for minimum pressure

PROPOSED PAVILION BEARING CAPACITY CALCULATIONS
SOIL BORING SS-6

MODULUS OF SUBGRADE REACTION (Scott 1981)

$K_s := 6 \cdot N_{cor} \cdot \frac{ton}{ft^3}$ $\boxed{K_s = 120000 \frac{lb}{ft^3}}$ *<~ Modulus of Subgrade Reaction*

ALLOWABLE BEARING RESISTANCE (Munfakh 2001)

$q_{n\_munfakh} := c \cdot N_c \cdot s_c + \gamma \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma \cdot C_{w\gamma}$

$q_{n\_munfakh} = 2947.7\ psf$

$\phi_b := 0.45$ *<~ Resistance Factor (LRFD)*

$q_{a\_munfakh} := \phi_b \cdot q_{n\_munfakh}$

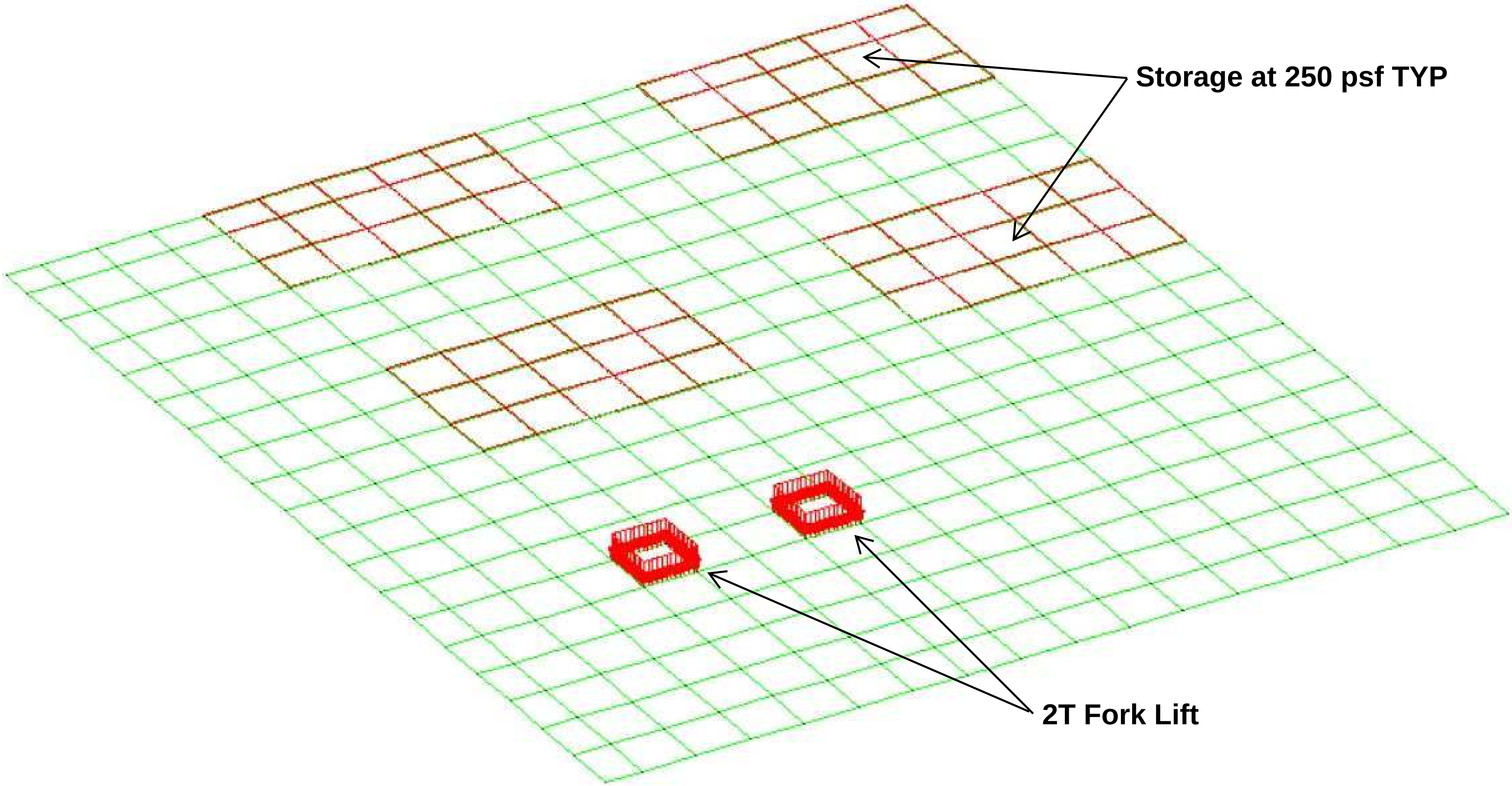
$q_{a\_munfakh} = 1326\ psf$ *<~ Allowable Bearing Resistance*

$B_{ftg} := 1ft + 4in = 1.333\ ft$

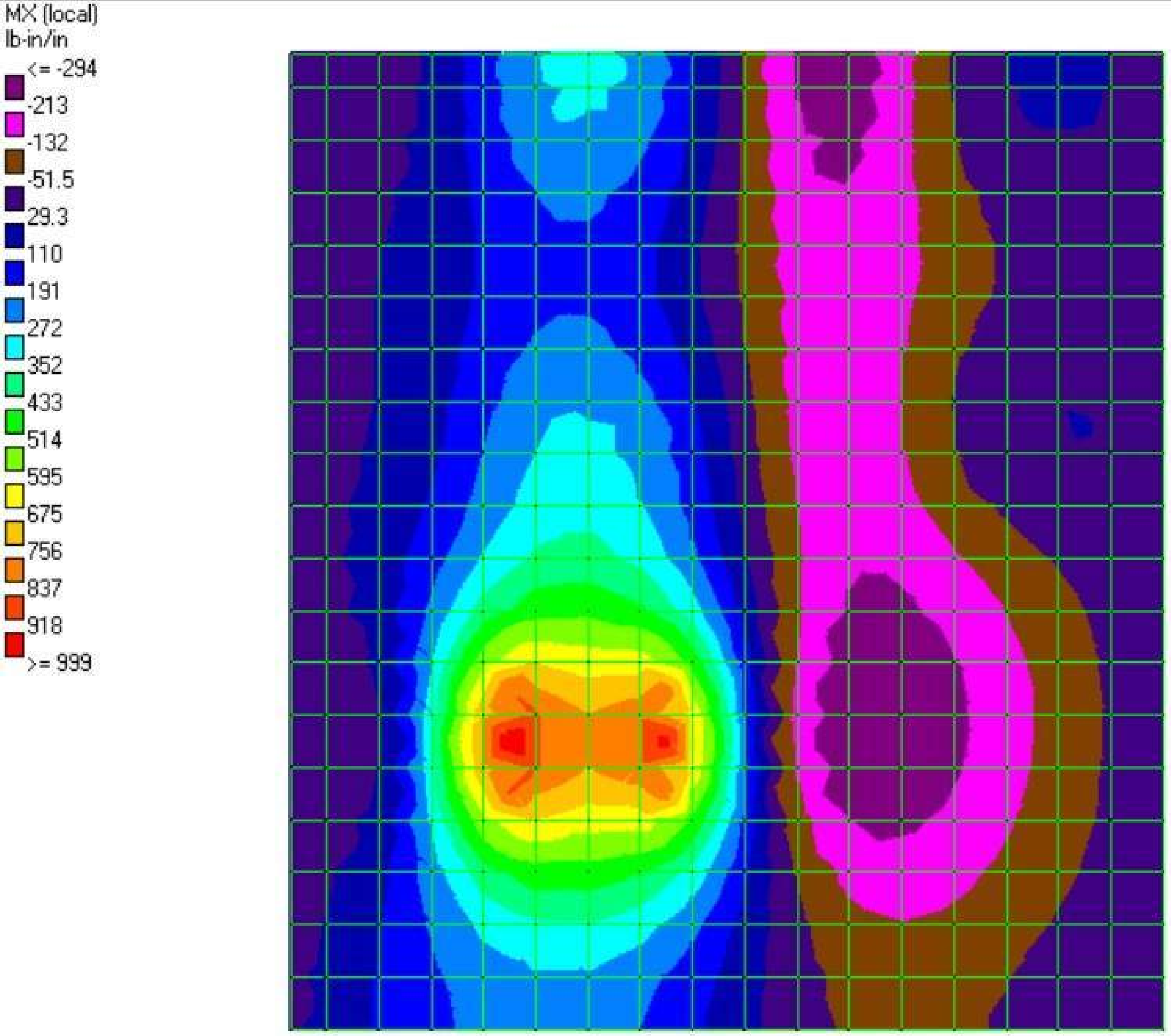
$A_{ftg} := 1ft \cdot B_{ftg} = 1.333 \cdot ft^2$ $S_{ftg} := \frac{1ft \cdot (B_{ftg})^2}{6} = 0.296 \cdot ft^3$

$f_{pLC6apmax} := \frac{1ft \cdot P_{LC3}}{A_{ftg}} = 982.500 \cdot psf$ $< f_{pmax} = 1326\ psf$ **OK**

$f_{pLC6apmin} := \frac{1ft \cdot P_{LC7n}}{A_{ftg}} = 298.775 \cdot psf$



SLAB LOADED AREA



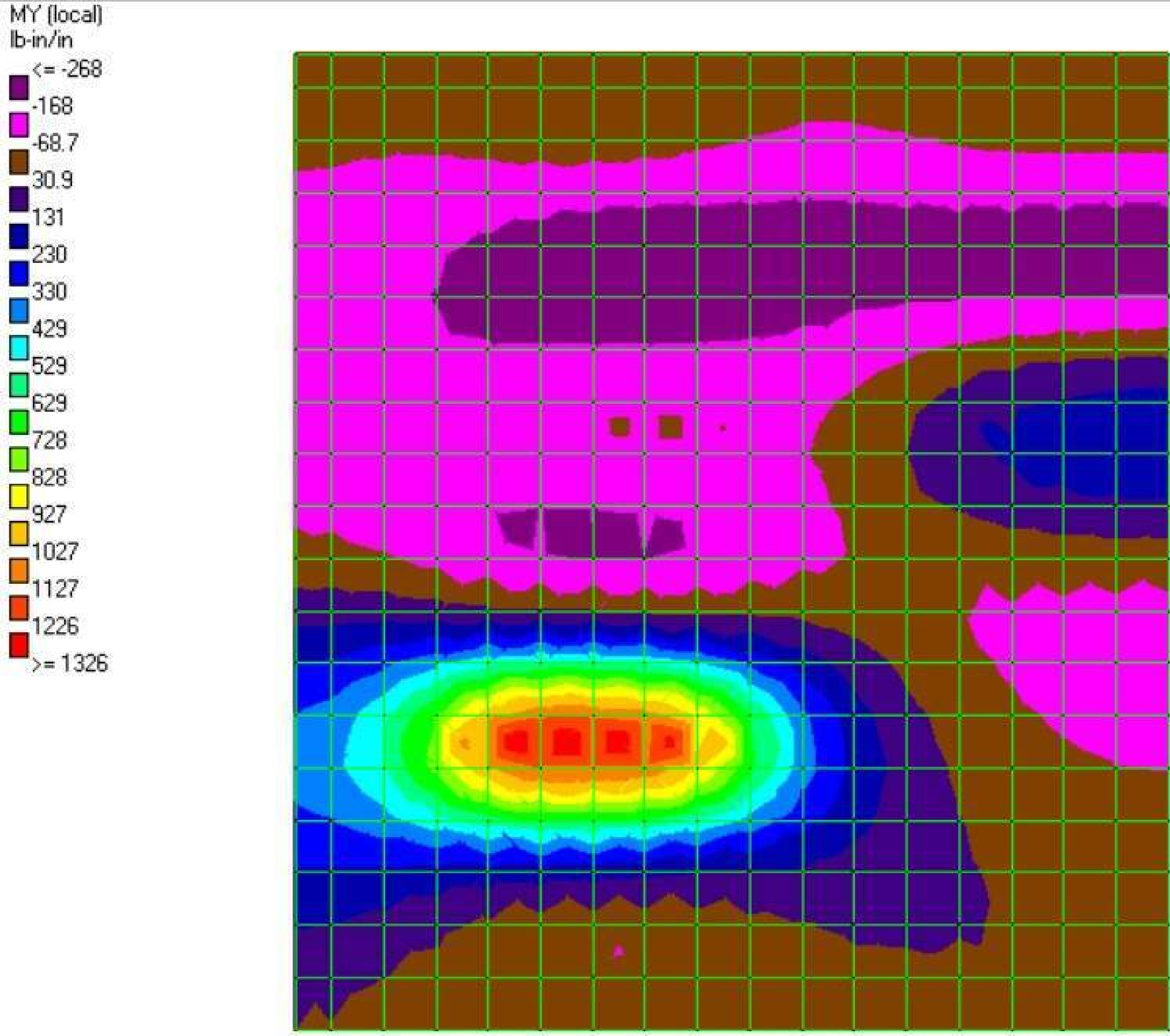
SLAB Mx

Loads applied are factored LRFD loads

Maximum Mx = 999 lb-in/in = 999 lb-ft/ft **A_s - 0.075 in^2/ft**

Minimum steel = 0.0018(12)(6) = 0.13 in^2/ft

Use #4 @ 18" A_s = 0.133 in^2/ft



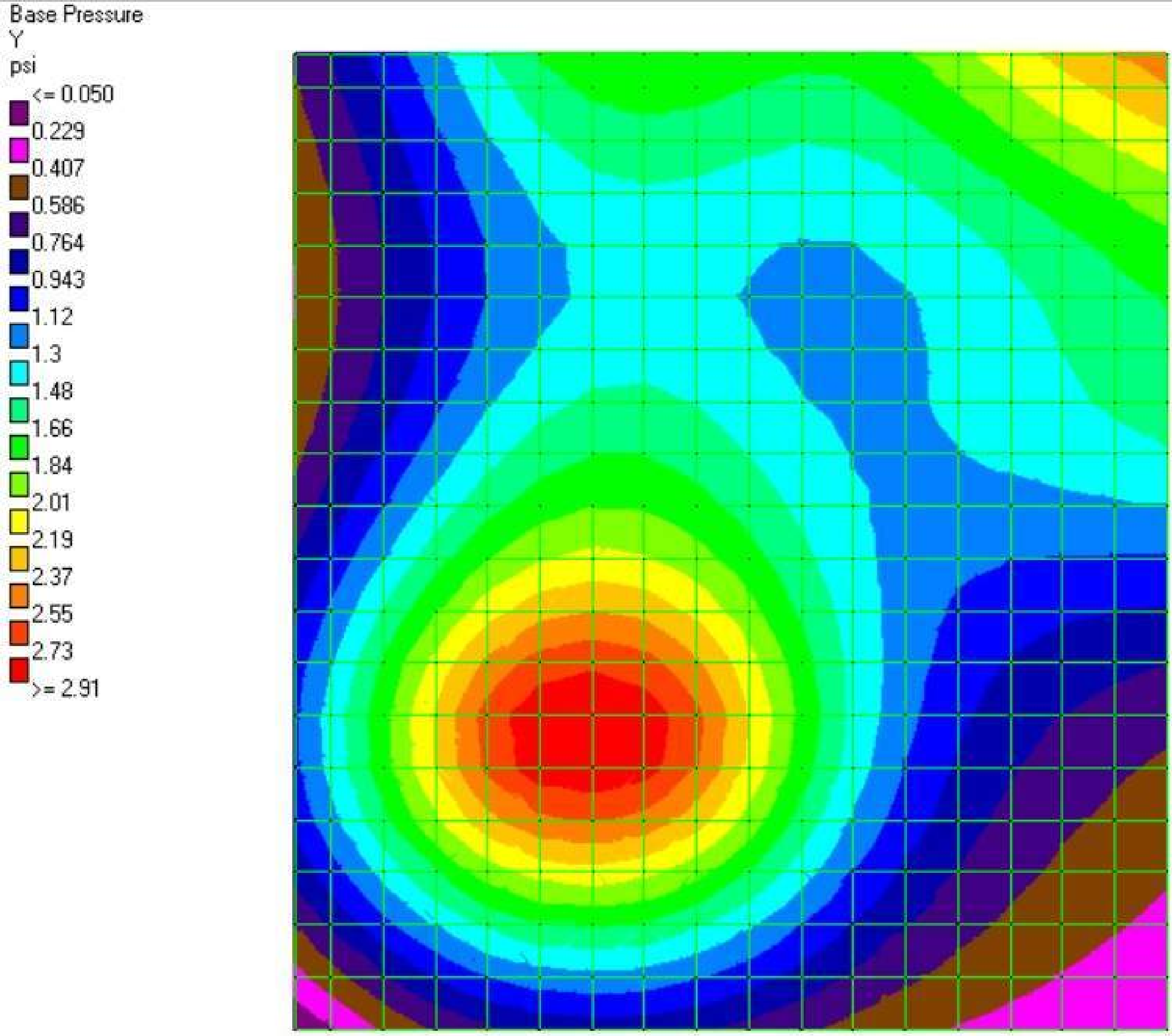
SLAB My

Loads applied are factored LRFD loads

Maximum Mx = 1326 lb-in/in = 1326 lb-ft/ft **A_s - 0.101 in^2/ft**

Minimum steel = 0.0018(12)(6) = 0.13 in^2/ft

Use #4 @ 18" A_s = 0.133 in^2/ft



SLAB BEARING PRESSURE

Maximum Pressure = 2.91 psi = 419 psf < 1326 psf OK

ATTACHMENT D
Dumpster Pavilion Design

gai consultants
transforming ideas into reality®

Tekla Tedds	Project I-10 Rest Area				Job Ref. B190988.00	
	Section Dumpter Area CMU Wall				Sheet no./rev. 1	
	Calc. by NJA	Date 5/14/2020	Chk'd by	Date	App'd by	Date

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the allowable stress design method

Tedds calculation version 2.1.02

Masonry wall panel details

Reinforced single-wythe wall, the wall is free at the top and fixed at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **18** ft

Panel height h = **8** ft

Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

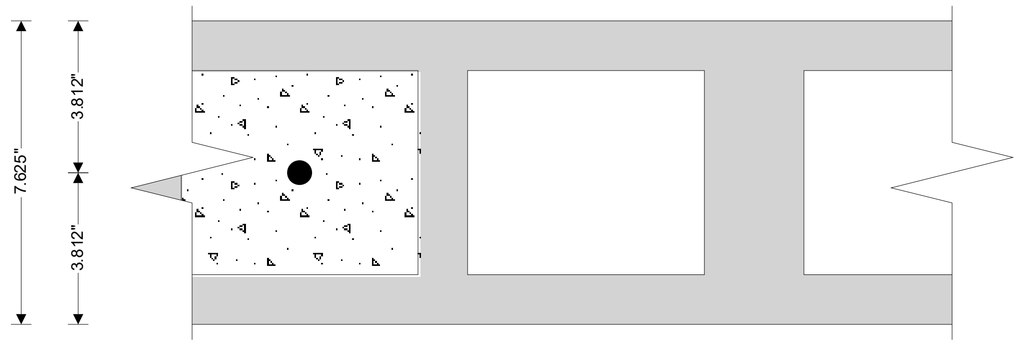
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

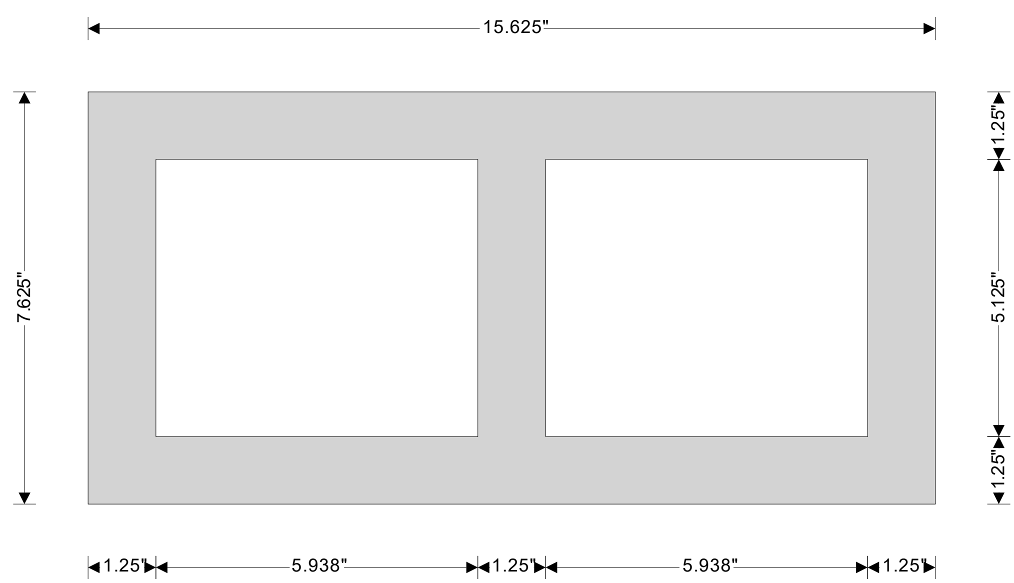
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit	f'_{cu} = **2800** psi
Density of masonry units	γ_{block} = **115** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From MSJC-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **2000** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1800000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **720000** psi

From MSJC-11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed F_{t_norm} = **51** psi

Allowable flexural tensile stress parallel to bed F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.15** in^2/ft

Lateral loading details

Wind load on panel W = **40.8** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Shear loading details

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.015)

Load combination no.5 DL + 0.6 × W (0.514)

Load combination no.7 DL + 0.75 × LL + 0.45 × W + 0.75 × (LL_r or SL or RL) (0.385)

Load combination no.9 0.6 × DL + 0.6 × W (0.529)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in^4/ft

Section modulus $S = I / c$ = **98.3** in^3/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

Effective height factor K = **1**

Consider wall at bottom under load combination no.9

Axial load at bottom of panel $P = 0.6 \times h \times w_{SW}$ = **244** lb/ft

Compressive stress due to axial load $f_a = P / A$ = **4** psi

Slenderness ratio $(K \times h) / r$ = **38.470** < 99

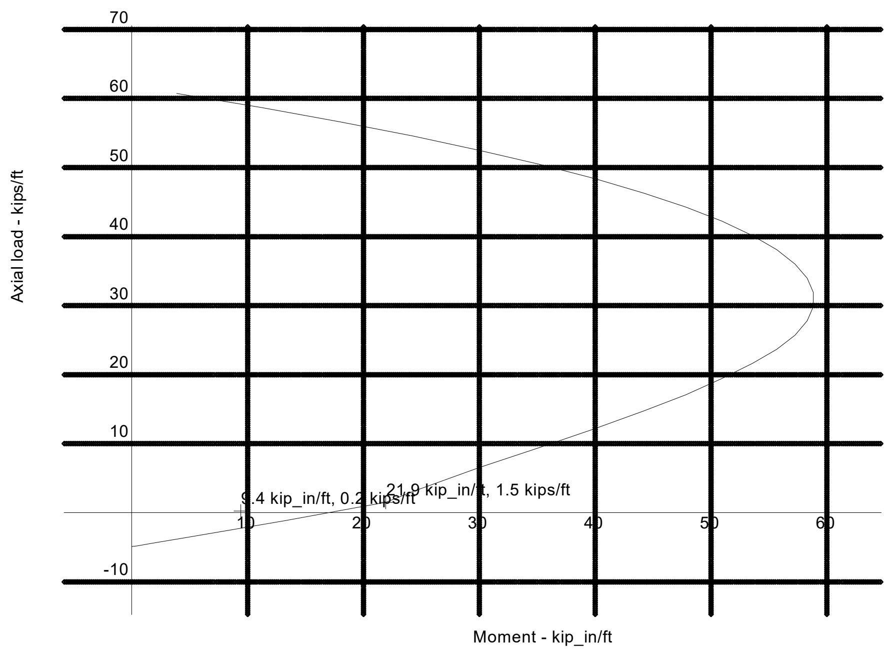
Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2]$ = **462.2** psi

f_a / F_a = **0.009**

Allowable compressive force $P_a = 0.25 \times f'_m \times A \times [1 - ((K \times h) / (140 \times r))^2]$ = **27820** lb/ft

P / P_a = **0.009**

Bending moment at bottom of panel	$M = 0.6 \times W \times h^2 / 2 = \mathbf{9400}$ lb_in/ft
Depth of reinforcement	$d = \mathbf{3.813}$ in
Modular ratio	$n = E_s / E_m = \mathbf{16.111}$
Allowable compressive stress due to flexural load	$F_b = (0.45) \times f'_m = \mathbf{900}$ psi
Balance point	$k_{bal} = n / (F_s / F_b + n) = \mathbf{0.312}$
Tensile strain in reinforcement	$\varepsilon_s = F_s / E_s = \mathbf{0.001103}$
Compressive strain in masonry	$\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) = \mathbf{0.000500}$
Compressive stress at balance point	$f_{bal} = \varepsilon_m \times E_m = \mathbf{900}$ psi
Tension at balance point	$T_{bal} = A_s \times F_s = \mathbf{4909}$ lb/ft
Compression at balance point	$C_{bal} = k_{bal} \times d \times f_{bal} / 2 = \mathbf{6420}$ lb/ft
Axial load at balance point	$P_{bal} = C_{bal} – T_{bal} = \mathbf{1511}$ lb/ft
Moment at balance point	$M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 – k_{bal} \times d / 3) = \mathbf{21931}$ lb_in/ft
Maximum moment from interaction diagram	$M_c = \mathbf{17760}$ lb_in/ft
	$M / M_c = \mathbf{0.529}$



Allowable stress interaction diagram

Consider wall at bottom under load combination no.9

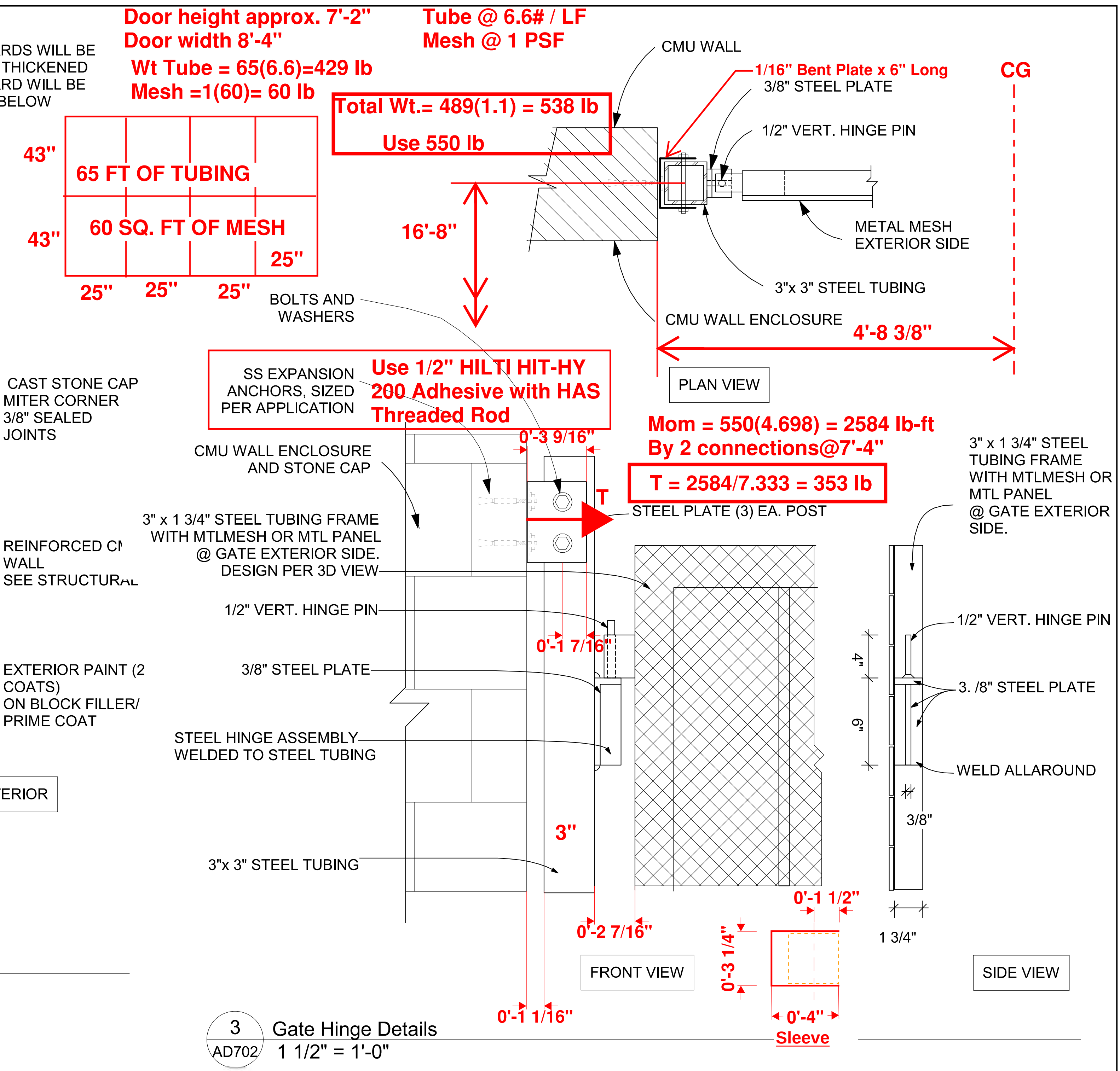
Shear force	$V = 0.6 \times W \times h = \mathbf{195.8}$ lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) = \mathbf{14.9}$ in^2/ft
Shear stress	$f_v = V / A_{nv} = \mathbf{13.2}$ psi
Compressive force	$N_v = 0.6 \times h \times w_{SW} = \mathbf{243.6}$ lb/ft
Moment	$M = 0.6 \times W \times h^2 / 2 = \mathbf{9400.3}$ lb_in/ft
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_v / A = \mathbf{51.3}$ psi

Allowable shear stress $F_v = \min(F_{vm},2 \times \sqrt{(f'_m \times 1\text{ psi})}) =$ **51.3** psi

Shear utilization $f_v / F_v =$ **0.256**

Dumpster Gate Connection

DUMPSTER GATE ATTACHMENT



3 AD702 Gate Hinge Details
1 1/2" = 1'-0"

PROJECT NORTH | 100% CONSTRUCTION DOCUMENTS

BORRELLI + PARTNERS
ARCHITECTURE PLANNING LANDSCAPE INTERIORS

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 8	COLUMBIA	438609-1-52-01

SR 8 (I-10) WESTBOUND - (COLUMBIA COUNTY) REST AREA

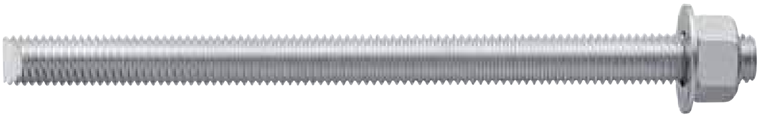
DUMPSTER DETAILS

DRAWING NO. AD702

SHEET NO.

HIT-HY 200 Adhesive with HAS Threaded Rod

GATE CONNECTION TO CMU BOND BEAM - ANCHORS
MAX TENSION: 180 LB MAX SHEAR: 140 LB



Hilti HAS threaded rod

Figure 9 - Hilti HAS threaded rod installation conditions

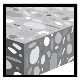
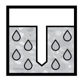
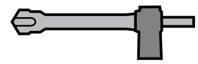
Permissible concrete conditions			Permissible drilling method	
	 Uncracked concrete	 Dry concrete		 Hammer drilling with carbide tipped drill bit
	 Cracked concrete	 Water saturated concrete		 Hilti TE-CD or TE-YD Hollow Drill Bit

Table 38 - Hilti HAS threaded rod specifications

Setting information		Symbol	Units	Nominal rod diameter, d 3/8	1/2	5/8	3/4	7/8	1	1-1/4
Nominal bit diameter		d_o	in.	7/16	9/16	3/4	7/8	1	1-1/8	1-3/8
Effective embedment	minimum	$h_{ef,min}$	in. (mm)	2-3/8 (60)	2-3/4 (70)	3-1/8 (79)	3-1/2 (89)	3-1/2 (89)	4 (102)	5 (127)
	maximum	$h_{ef,max}$	in. (mm)	7-1/2 (191)	10 (254)	12-1/2 (318)	15 (381)	17-1/2 (445)	20 (508)	25 (635)
Diameter of fixture hole	through-set		in.	1/2	5/8	13/16[1]	15/16[1]	1-1/8[1]	1-1/4[1]	1-1/2[1]
Diameter of fixture hole	preset		in.	7/16	9/16	11/16	13/16	15/16	1-1/8	1-3/8
Installation torque		T_{inst}	ft-lb (Nm)	15 (20)	30 (40)	60 (80)	100 (136)	125 (169)	150 (203)	200 (271)
Minimum concrete thickness		h_{min}	in. (mm)	h_{ef}+1-1/4 (h_{ef}+30)	$h_{ef}+2d_o$					
Minimum edge distance		c_{min}	in. (mm)	1-3/4 (45)	1-3/4 (45)	2[2] (50)[2]	2-1/8[2] (55)[2]	2-1/4[2] (60)[2]	2-3/4[2] (70)[2]	3-1/8[2] (80)[2]
Minimum anchor spacing		s_{min}	in. (mm)	1-7/8 (48)	2-1/2 (64)	3-1/8 (79)	3-3/4 (95)	4-3/4 (111)	5 (127)	6-1/4 (159)

1 Install using (2) washers. See Figure 11.
2 Edge distance of 1-3/4-inch (44mm) is permitted provided the installation torque is reduced to 0.30 T_{inst} for 5d < s < 16-in. and to 0.5 T_{inst} for s > 16-in.

Figure 10 - Hilti HAS threaded rods

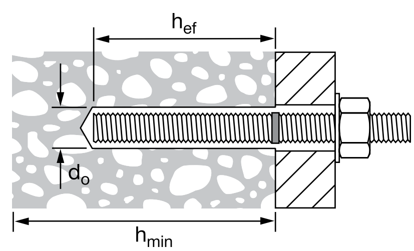


Figure 11 - Installation with (2) washers

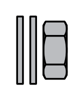


Table 39 - Hilti HIT-HY 200 adhesive design strength with concrete / bond failure for threaded rod in uncracked concrete [1,2,3,4,5,6,7,8,9]

Nominal anchor diameter in.	Effective embedment in. (mm)	Tension — ΦN_n				Shear — ΦV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
3/8	2-3/8 (60)	2,855 (12.7)	3,125 (13.9)	3,610 (16.1)	4,405 (19.6)	3,075 (13.7)	3,370 (15.0)	3,890 (17.3)	4,745 (21.1)
	3-3/8 (86)	4,835 (21.5)	5,300 (23.6)	6,015 (26.8)	6,260 (27.8)	10,415 (46.3)	11,410 (50.8)	12,950 (57.6)	13,490 (60.0)
	4-1/2 (114)	7,445 (33.1)	7,790 (34.7)	8,020 (35.7)	8,350 (37.1)	16,035 (71.3)	16,780 (74.6)	17,270 (76.8)	17,985 (80.0)
	7-1/2 (191)	12,750 (56.7)	12,985 (57.8)	13,365 (59.5)	13,915 (61.9)	27,460 (122.1)	27,965 (124.4)	28,785 (128.0)	29,975 (133.3)
1/2	2-3/4 (70)	3,555 (15.8)	3,895 (17.3)	4,500 (20.0)	5,510 (24.5)	7,660 (34.1)	8,395 (37.3)	9,690 (43.1)	11,870 (52.8)
	4-1/2 (114)	7,445 (33.1)	8,155 (36.3)	9,420 (41.9)	11,135 (49.5)	16,035 (71.3)	17,570 (78.2)	20,285 (90.2)	23,980 (106.7)
	6 (152)	11,465 (51.0)	12,560 (55.9)	14,255 (63.4)	14,845 (66.0)	24,690 (109.8)	27,045 (120.3)	30,700 (136.6)	31,970 (142.2)
	10 (254)	22,665 (100.8)	23,085 (102.7)	23,755 (105.7)	24,740 (110.0)	48,820 (217.2)	49,720 (221.2)	51,170 (227.6)	53,285 (237.0)
5/8	3-1/8 (79)	4,310 (19.2)	4,720 (21.0)	5,450 (24.2)	6,675 (29.7)	9,280 (41.3)	10,165 (45.2)	11,740 (52.2)	14,380 (64.0)
	5-5/8 (143)	10,405 (46.3)	11,400 (50.7)	13,165 (58.6)	16,120 (71.7)	22,415 (99.7)	24,550 (109.2)	28,350 (126.1)	34,720 (154.4)
	7-1/2 (191)	16,020 (71.3)	17,550 (78.1)	20,265 (90.1)	23,195 (103.2)	34,505 (153.5)	37,800 (168.1)	43,650 (194.2)	49,955 (222.2)
	12-1/2 (318)	34,470 (153.3)	36,070 (160.4)	37,120 (165.1)	38,655 (171.9)	74,245 (330.3)	77,685 (345.6)	79,955 (355.7)	83,260 (370.4)
3/4	3-1/2 (89)	5,105 (22.7)	5,595 (24.9)	6,460 (28.7)	7,910 (35.2)	11,000 (48.9)	12,050 (53.6)	13,915 (61.9)	17,040 (75.8)
	6-3/4 (171)	13,680 (60.9)	14,985 (66.7)	17,305 (77.0)	21,190 (94.3)	29,460 (131.0)	32,275 (143.6)	37,265 (165.8)	45,645 (203.0)
	9 (229)	21,060 (93.7)	23,070 (102.6)	26,640 (118.5)	32,625 (145.1)	45,360 (201.8)	49,690 (221.0)	57,375 (255.2)	70,270 (312.6)
	15 (381)	45,315 (201.6)	49,640 (220.8)	53,455 (237.8)	55,665 (247.6)	97,600 (434.1)	106,915 (475.6)	115,130 (512.1)	119,895 (533.3)
7/8	3-1/2 (89)	5,105 (22.7)	5,595 (24.9)	6,460 (28.7)	7,910 (35.2)	11,000 (48.9)	12,050 (53.6)	13,915 (61.9)	17,040 (75.8)
	7-7/8 (200)	17,235 (76.7)	18,885 (84.0)	21,805 (97.0)	26,705 (118.8)	37,125 (165.1)	40,670 (180.9)	46,960 (208.9)	57,515 (255.8)
	10-1/2 (267)	26,540 (118.1)	29,070 (129.3)	33,570 (149.3)	41,115 (182.9)	57,160 (254.3)	62,615 (278.5)	72,300 (321.6)	88,550 (393.9)
	17-1/2 (445)	57,100 (254.0)	62,550 (278.2)	72,230 (321.3)	75,770 (337.0)	122,990 (547.1)	134,730 (599.3)	155,570 (692.0)	163,190 (725.9)
1	4 (102)	6,240 (27.8)	6,835 (30.4)	7,895 (35.1)	9,665 (43.0)	13,440 (59.8)	14,725 (65.5)	17,000 (75.6)	20,820 (92.6)
	9 (229)	21,060 (93.7)	23,070 (102.6)	26,640 (118.5)	32,625 (145.1)	45,360 (201.8)	49,690 (221.0)	57,375 (255.2)	70,270 (312.6)
	12 (305)	32,425 (144.2)	35,520 (158.0)	41,015 (182.4)	50,230 (223.4)	69,835 (310.6)	76,500 (340.3)	88,335 (392.9)	108,190 (481.3)
	20 (508)	69,765 (310.3)	76,425 (340.0)	88,245 (392.5)	98,960 (440.2)	150,265 (668.4)	164,605 (732.2)	190,070 (845.5)	213,150 (948.1)
1-1/4	5 (127)	8,720 (38.8)	9,555 (42.5)	11,030 (49.1)	13,510 (60.1)	18,785 (83.6)	20,575 (91.5)	23,760 (105.7)	29,100 (129.4)
	11-1/4 (286)	29,430 (130.9)	32,240 (143.4)	37,230 (165.6)	45,595 (202.8)	63,395 (282.0)	69,445 (308.9)	80,185 (356.7)	98,205 (436.8)
	15 (381)	45,315 (201.6)	49,640 (220.8)	57,320 (255.0)	70,200 (312.3)	97,600 (434.1)	106,915 (475.6)	123,455 (549.2)	151,200 (672.6)
	25 (635)	97,500 (433.7)	106,805 (475.1)	123,330 (548.6)	151,045 (671.9)	210,000 (934.1)	230,045 (1023.3)	265,630 (1181.6)	325,330 (1447.1)

1 See section 3.1.8 for explanation on development of load values.
2 See section 3.1.8 to convert design strength (factored resistance) value to ASD value.
3 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.
4 Apply spacing, edge distance, and concrete thickness factors in tables 42 - 55 as necessary to the above values. Compare to the steel values in table 41. The lesser of the values is to be used for the design.
5 Data is for temperature range A: Max. short term temperature = 130° F (55° C), max. long term temperature = 110° F (43° C).
For temperature range B: Max. short term temperature = 176° F (80° C), max. long term temperature = 110° F (43° C) multiply above values by 0.92.
For temperature range C: Max. short term temperature = 248° F (120° C), max. long term temperature = 162° F (72° C) multiply above values by 0.78.
Short term elevated concrete temperatures are those that occur over brief intervals, e.g., as a result of diurnal cycling. Long term concrete temperatures are roughly constant over significant periods of time.
6 Tabular values are for dry and water saturated concrete conditions.
7 Tabular values are for short term loads only. For sustained loads including overhead use, see section 3.1.8.
8 Tabular values are for normal-weight concrete only. For lightweight concrete, multiply design strength (factored resistance) by λ_a as follows:
For sand-lightweight, λ_a = 0.51. For all-lightweight, λ_a = 0.45.
9 Tabular values are for static loads only. Seismic design is not permitted for uncracked concrete.

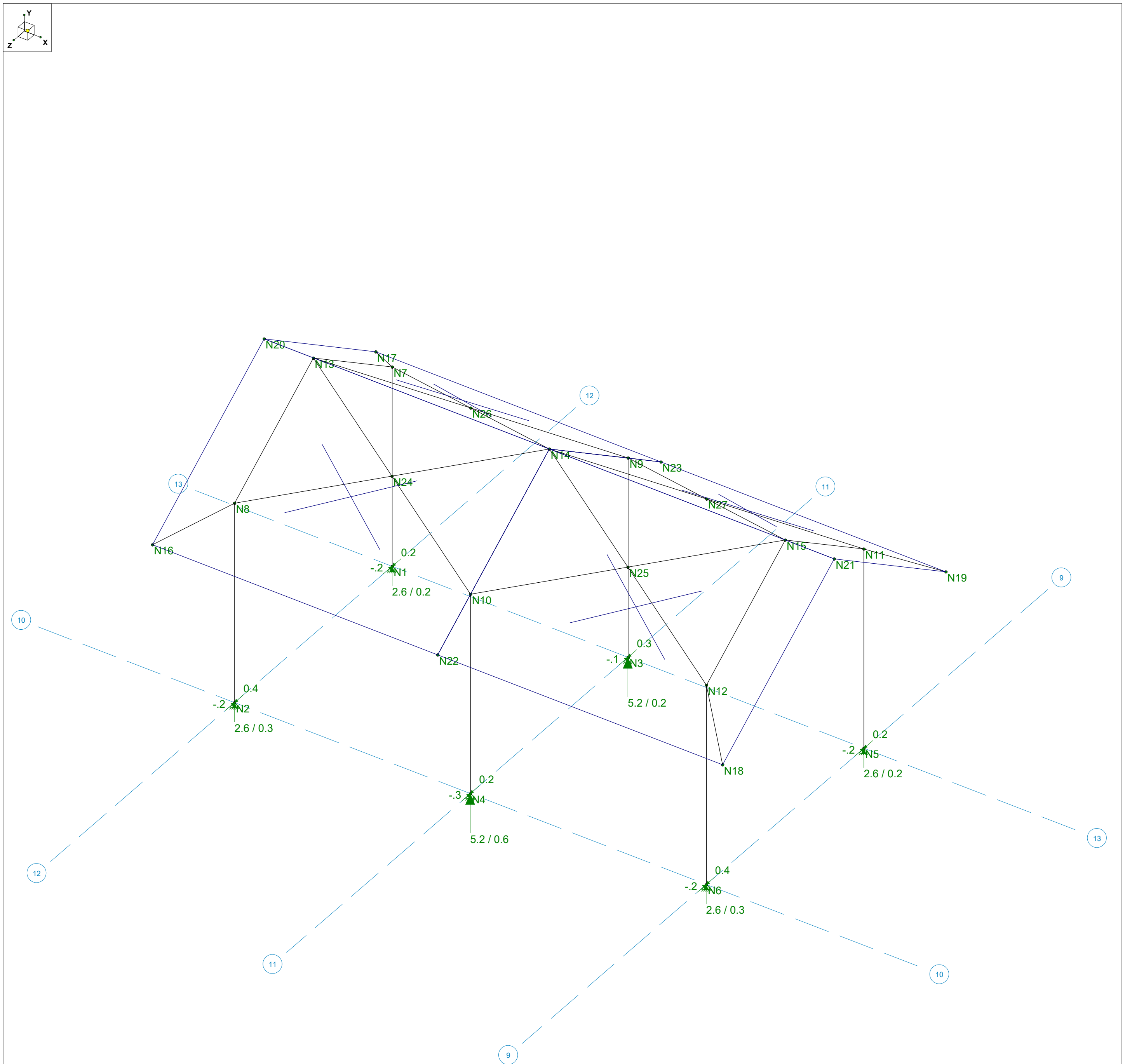
Table 41 - Steel design strength for Hilti HAS threaded rods for use with ACI 318-14 Chapter 17

Nominal anchor diameter in.	HAS-V-36 / HAS-V-36 HDG ASTM F1554 Gr. 55[4,5]			HAS-E-55 / HAS-E-55 HDG ASTM F1554 Gr. 55[4,5,6]			HAS-B-105 and HAS-B-105 HDG ASTM A193 B7 and ASTM F 1554 Gr.105[4]			HAS-R stainless steel ASTM F593 (3/8-in to 1-in)[5] ASTM A193 (1-1/8-in to 2-in)[4]		
	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)	Tensile[1] ΦN_{sa} lb (kN)	Shear[2] ΦV_{sa} lb (kN)	Seismic Shear[3] $\Phi V_{sa,eq}$ lb (kN)
3/8	3,370 (15.0)	1,750 (7.8)	1,050 (4.7)	4,360 (19.4)	2,270 (10.1)	1,590 (7.1)	7,270 (32.3)	3,780 (16.8)	2,645 (11.8)	5,040 (22.4)	2,790 (12.4)	1,955 (8.7)
1/2	6,175 (27.5)	3,210 (14.3)	1,925 (8.6)	7,985 (35.5)	4,150 (18.5)	2,905 (12.9)	13,305 (59.2)	6,920 (30.8)	4,845 (21.6)	9,225 (41.0)	5,110 (22.7)	3,575 (15.9)
5/8	9,835 (43.7)	5,110 (22.7)	3,065 (13.6)	12,715 (56.6)	6,610 (29.4)	4,625 (20.6)	21,190 (94.3)	11,020 (49.0)	7,715 (34.3)	14,690 (65.3)	8,135 (36.2)	5,695 (25.3)
3/4	14,550 (64.7)	7,565 (33.7)	4,540 (20.2)	18,820 (83.7)	9,785 (43.5)	6,850 (30.5)	31,360 (139.5)	16,310 (72.6)	11,415 (50.8)	18,485 (82.2)	10,235 (45.5)	7,165 (31.9)
7/8	20,085 (89.3)	10,445 (46.5)	6,265 (27.9)	25,975 (115.5)	13,505 (60.1)	9,455 (42.1)	43,285 (192.5)	22,510 (100.1)	15,755 (70.1)	25,510 (113.5)	14,125 (62.8)	9,890 (44.0)
1	26,350 (117.2)	13,700 (60.9)	8,220 (36.6)	34,075 (151.6)	17,720 (78.8)	12,405 (55.2)	56,785 (252.6)	29,530 (131.4)	20,670 (91.9)	33,465 (148.9)	18,535 (82.4)	12,975 (57.7)
1-1/4	42,160 (187.5)	21,920 (97.5)	13,150 (58.5)	54,515 (242.5)	28,345 (126.1)	19,840 (88.3)	90,855 (404.1)	47,245 (210.2)	33,070 (147.1)	41,430 (184.3)	21,545 (95.8)	12,925 (57.5)

1 Tensile = $\phi A_{se,N} f_{uta}$ as noted in ACI 318-14 17.4.1.2
2 Shear = $\phi\ 0.60\ A_{se,V} f_{uta}$ as noted in ACI 318-14 17.5.1.2b.
3 Seismic Shear = $\alpha_{V,seis}\ \phi\ V_{sa}$: Reduction factor for seismic shear only. See ACI 318 for additional information on seismic applications.
4 HAS-V, HAS-E (3/8-in to 1-1/4-in), HAS-B, and HAS-R (Class 1; 1-1/4-in) threaded rods are considered ductile steel elements (including HDG rods).
5 HAS-R (CW1 and CW2; 3/8-in to 1-in) threaded rods are considered brittle steel elements.
6 3/8-inch dia. threaded rods are not included in the ASTM F1554 standard. Hilti 3/8-inch dia. HAS-V, HAS-E, and HAS-E-B (incl. HDG) threaded rods meet the chemical composition and mechanical property requirements of ASTM F1554.

ATTACHMENT E
Picnic Pavilion Design

gai consultants
transforming ideas into reality®



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

GAI Consultants	I-10 Picnic Pavilion	SK - 1
NJA		July 7, 2020 at 9:30 AM
B190988		Model for loads.r3d

Plate Surface Loads (BLC 1 : Dead)

	Plate Label	Direction	Magnitude[ksf,F]
1	P1	Y	-.01
2	P2	Y	-.01
3	P3	Y	-.01
4	P4	Y	-.01

Plate Surface Loads (BLC 2 : Live)

	Plate Label	Direction	Magnitude[ksf,F]
1	P1	Y	-.02
2	P2	Y	-.02
3	P3	Y	-.02
4	P4	Y	-.02

Plate Surface Loads (BLC 3 : Wind Case A)

	Plate Label	Direction	Magnitude[ksf,F]
1	P1	z	-.005
2	P2	z	-.005
3	P3	z	-.01
4	P4	z	-.01

Plate Surface Loads (BLC 4 : Wind Case B)

	Plate Label	Direction	Magnitude[ksf,F]
1	P1	z	.016
2	P2	z	.016
3	P3	z	-.008
4	P4	z	-.008

Basic Load Cases

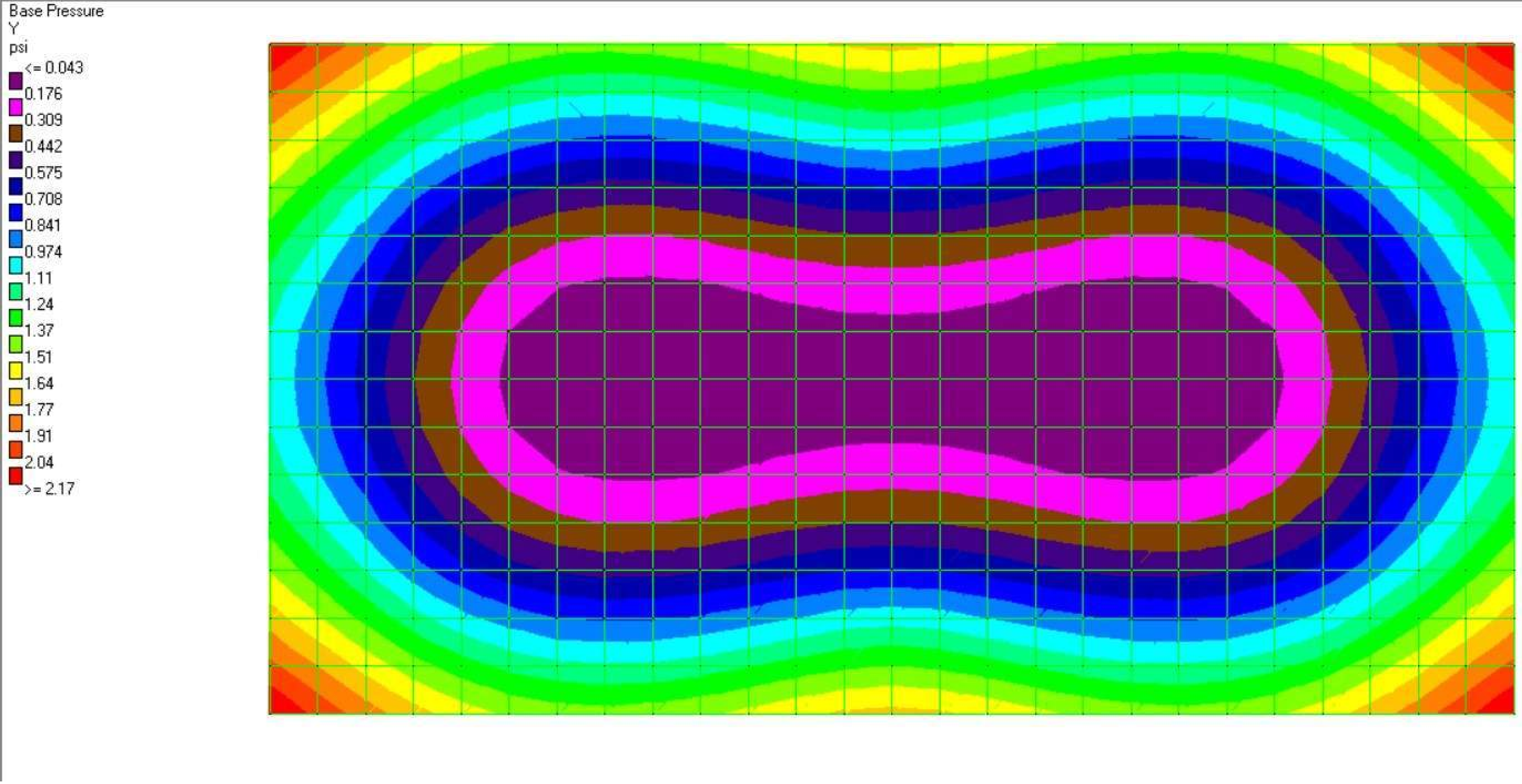
	BLC Description	Category	X Gr...	Y Gra...	Z Gra...	Joint	Point	Distrib...	Area(Member)	Surface(Plate/Wall)
1	Dead	DL		-1						4
2	Live	LL								4
3	Wind Case A	WL								4
4	Wind Case B	WL								4

Load Combinations

	Description	S...	P...	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	BLC	Fa...
1	1	Yes			DL	1																		
2	2	Yes			DL	1	LL	1																
3	6	Yes			DL	1	LL	.75	3	.45														
4	6	Yes			DL	1	LL	.75	4	.45														
5	7	Yes			DL	.6	LL		3	.6														
6	7	Yes			DL	.6	LL		4	.6														

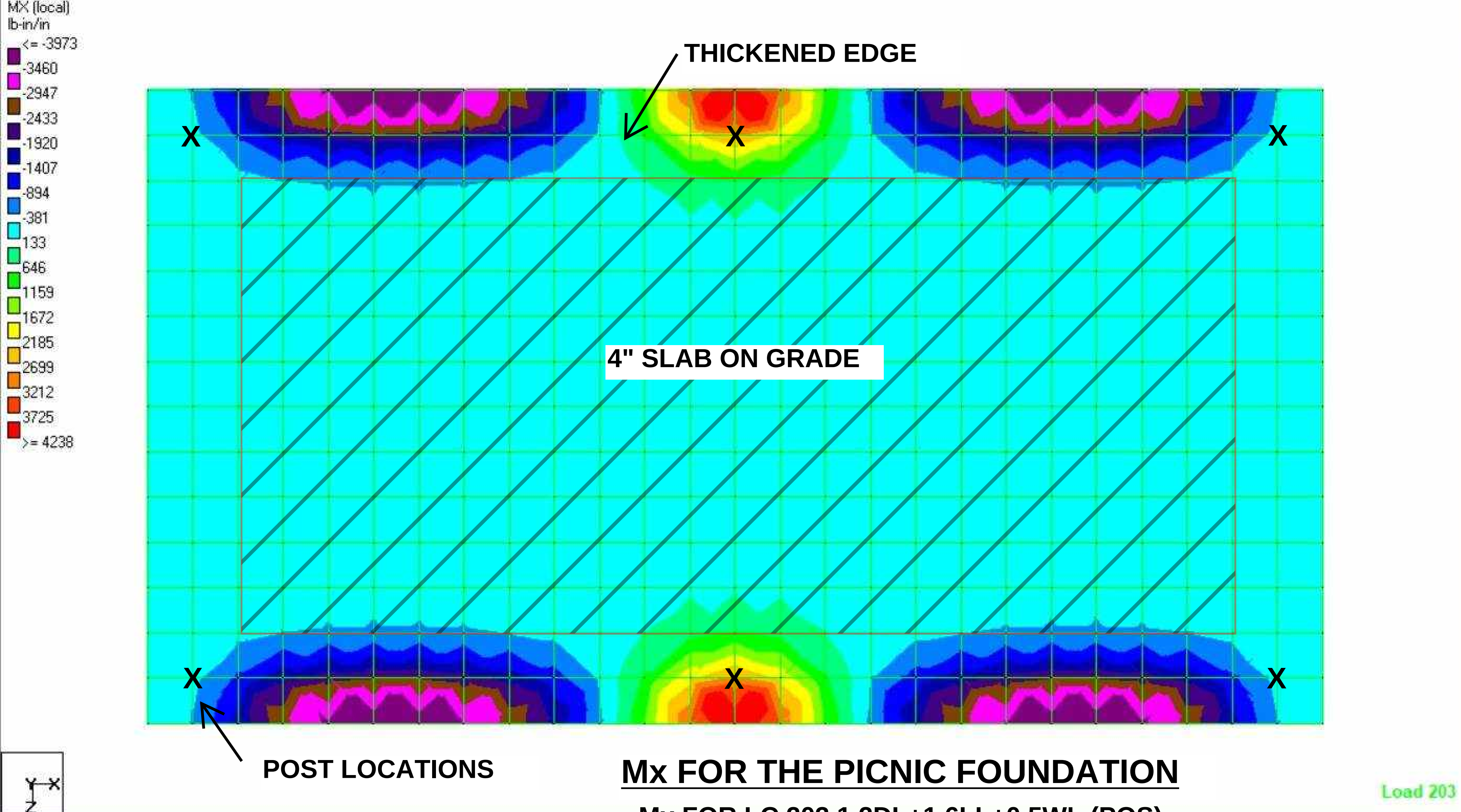
Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	-.02	6	2.562	2	.355	3	0	6	0	6	0	6
2		min	-.242	2	.29	6	-.164	6	0	1	0	1	0	1
3	N4	max	0	6	5.207	2	.196	5	0	6	0	6	0	6
4		min	0	5	.625	6	-.259	4	0	1	0	1	0	1
5	N1	max	-.021	6	2.562	2	.216	5	0	6	0	6	0	6
6		min	-.242	2	.163	6	-.221	4	0	1	0	1	0	1
7	N3	max	0	5	5.207	2	.33	3	0	6	0	6	0	6
8		min	0	6	.206	6	-.063	6	0	1	0	1	0	1
9	N6	max	.242	2	2.562	2	.355	3	0	6	0	6	0	6
10		min	.02	6	.29	6	-.164	6	0	1	0	1	0	1
11	N5	max	.242	2	2.562	2	.216	5	0	6	0	6	0	6
12		min	.021	6	.163	6	-.221	4	0	1	0	1	0	1
13	Totals:	max	0	6	20.662	2	1.479	5						
14		min	0	5	1.738	6	-.789	6						



PICNIC FOUNDATION BASE PRESSURE

MAXIMUM PRESSURE = 2.17 psi = 312.5 psf



Mx FOR THE PICNIC FOUNDATION

Mx FOR LC 203 1.2DL+1.6LL+0.5WL (POS)

Mx =-3973 lb-in/in = -3973 lb-ft/ft (TOP STL)

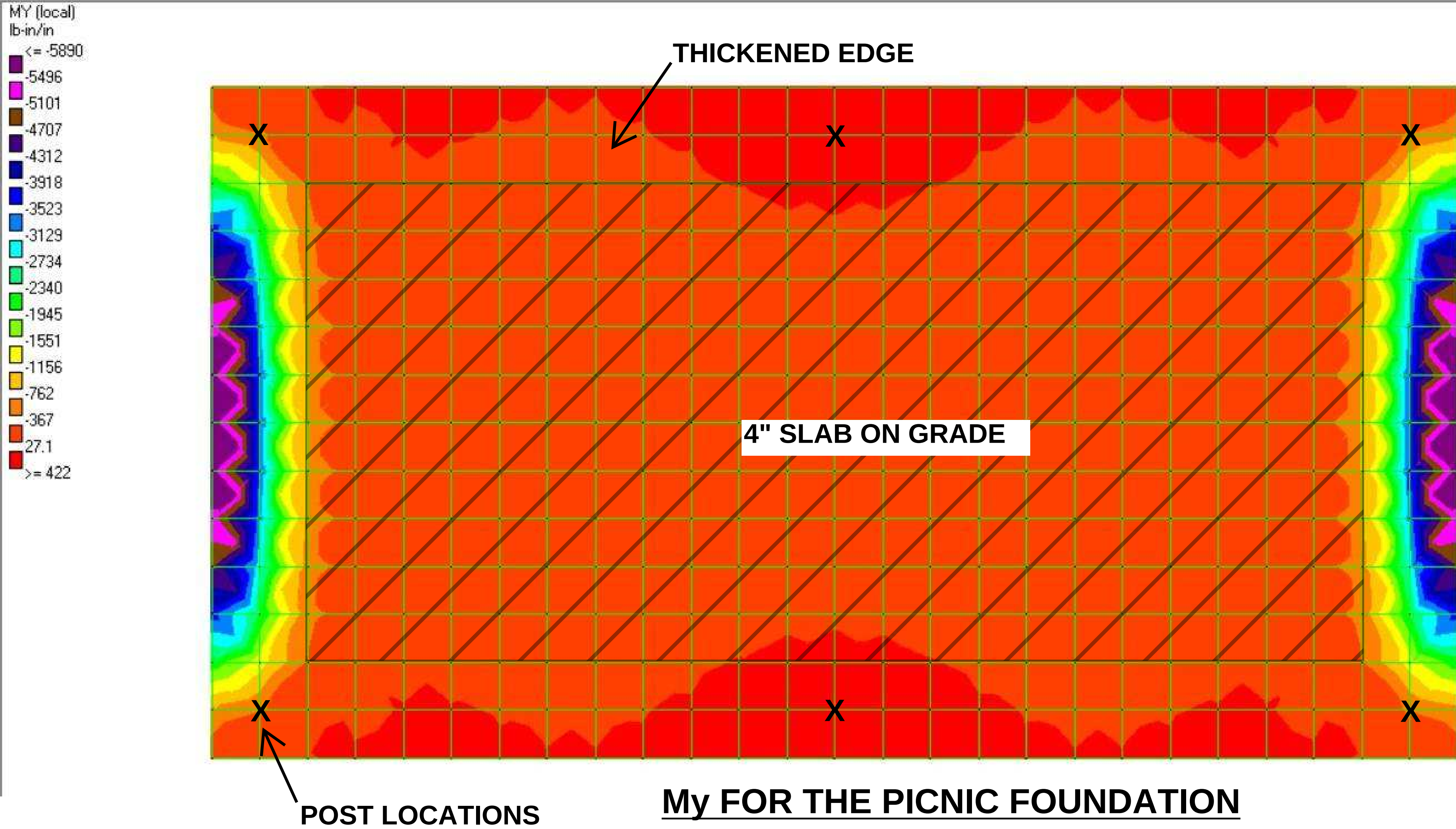
Mx =+4238 lb-in/in = 4238 lb-ft/ft (BOT.STL)

16 " THICKENED EDGE

Mx =-0 lb-in/in = -0 lb-ft/ft (TOP STL)

Mx = +646 lb-in/in = 646 lb-ft/ft (BOT.STL)

4 " SLAB



My FOR THE PICNIC FOUNDATION

My FOR LC 203 1.2DL+1.6LL+0.5WL (POS)

My =-5890 lb-in/in = -5890 lb-ft/ft (TOP STL)

My = +422 lb-in/in = 422 lb-ft/ft (BOT.STL)

$\}$ **16 " THICKENED EDGE**

My =-0 lb-in/in = -0 lb-ft/ft (TOP STL)

My = +27.1 lb-in/in = 27.1 lb-ft/ft (BOT.STL)

$\}$ **4 " SLAB**