Building Notes

1) THESE STRUCTURAL DRAWINGS SHALL BE USED FOR THE CONSTRUCTION Of THE SHOWN STORAGE BUILDINGS. IN THE EVENT Of DIMENSIONAL DISCREPANCY, NOTIFY THE ENGINEER FOR RESOLUTION OF CONFLICT. ANY DEVIATION FROM THESE DRAWINGS MUST BE APPROVED BY THE ENGINEER.
2) THIS IS TO CERTIFY THAT THE WOOD FRAME STORAGE BUILDING AS SHOWN ON THESE DRAWINGS HAS BEEN DESIGNED IN ACCORDANCE WITH THE 2020 FLORIDA BUILDING CODE WITH ALL REVISIONS TO ASCE 7-16 FOR UP TO 175 MPH WIND VELOCITY.
3) MATERIALS.
   ALL VERTICAL LUMBER TO BE NO 2 GRADE SPF OR EQUIVALENT.
   ALL HORIZONTAL FLOOR FRAMING TO BE NO 2 GRADE SYP. ALL PLYWOOD/PLYWOOD GUSSETS SHALL CONFORM TO APA PDS-04.
   ALL CONNECTORS/FASTENERS (SIMPSON STRONG TIE OR APPROVED) APPLICABLE CODES, TO ASSURE SUPPORT.
   * NO 2 GRADE SPF LUMBER OR EQUIVALENT IS APPLICABLE UP TO 155MPH SPEED. FOR SPEEDS HIGHER THAN 155MPH USE NO 2 GRADE SYP OR EQUIVALENT FOR SPEEDS UP TO 175MPH.
4) DIMENSIONS LABELED BY LETTERS VARY BY MODEL AND SIZE OF BUILDING AND SHALL BE OBTAINED FROM THE PROVIDED SCHEDULES.
5) NON-STRUCTURAL DETAILS AND ITEMS MAY BE CHANGED AT OWNERS DISCRETION.
6) ALL FASTENERS AND CONNECTORS IN CONTACT WITH PRESSURE TREATED WOOD SHALL BE HOT DIP GALVANIZED (G 185) OR STAINLESS STEEL. ALL LUMBER IN CONTACT WITH THE EARTH SHALL BE PRESSURE TREATED WITH PRESERVATIVE. EXTERIOR NON-TREATED WOOD SIDING SHALL NOT BE LESS THAN 12" FROM EXPOSED EARTH.
7) ALL SITE WORK, INCLUDING BUT NOT LIMITED TO, CONCRETE/ANCHORING/ELECTRICAL CONNECTIONS, SHALL BE BY OTHERS.

Simpson Connector Notes

1) SIMPSON CONNECTIONS SPECIFIED ARE DESIGNED AND MANUFACTURED FOR THE PURPOSES SHOWN, AND SHOULD NOT BE USED WITH OTHER CONNECTORS NOT APPROVED BY THE DESIGN ENGINEER. MODIFICATIONS TO PRODUCTS OR CHANGES IN INSTALLATION PROCEDURES SHOULD NOT BE MADE WITHOUT THE APPROVAL OF THE ENGINEER. THE PERFORMANCE OF SUCH MODIFIED PRODUCTS OR ALTERED INSTALLATION PROCEDURES IS THE SOLE RESPONSIBILITY OF THE OWNER/CONTRACTOR.
2) SUBSTITUTIONS FOR SIMPSON STRONG-TIE CO. INC'S PRODUCTS SHALL BE APPROVED IF EQUAL AND APPROVED IN WRITING BY THE ENGINEER.

INSTRUCTIONS FOR THE INSTALLER:

1) ALL SPECIFIED FASTENERS MUST BE INSTALLED ACCORDING TO THE INSTRUCTIONS IN THE SIMPSON CATALOG. INCORRECT FASTENER QUANTITY, SIZE, TYPE, MATERIAL, OR FINISH MAY CAUSE THE CONNECTION TO FAIL. 16D FASTENERS ARE COMMON NAILS (18GA X 3-1/4 ") UNLESS OTHERWISE SPECIFIED.
2) BOLT HOLES SHALL BE A MINIMUM OF 1/32" AND A MAXIMUM OF 1/16" LARGER THAN THE BOLT DIAMETER.
3) INSTALL ALL SPECIFIED FASTENERS BEFORE LOADING THE CONNECTION.
4) PNEUMATIC OR POWDER-ACTUATED FASTENERS MAY DEFLECT AND INJURE THE OPERATOR OR OTHERS. NAIL GUNS MAY BE USED TO INSTALL CONNECTORS, PROVIDED THE CORRECT QUANTITY AND TYPE OF NAILS ARE PROPERLY INSTALLED IN THE NAIL HOLES. GUNS WITH NAIL HOLE-LOCATING MECHANISMS SHOULD BE USED. FOLLOW THE MANUFACTURERS INSTRUCTIONS AND USE THE APPROPRIATE SAFETY EQUIPMENT.

Structural Notes

STRUCTURAL DESIGN IS IN ACCORDANCE WITH FBC 2020 7TH EDITION

General Notes

1) 1) THIS COVER SHEET AND ADDITIONAL ACCOMPANYING ATTACHMENT SHEETS REPRESENT MINIMUM DESIGN REQUIREMENTS FOR CONSTRUCTION OF THE ATTACHED SEALED PLANS IN ACCORDANCE WITH ASCE 7-16, FOR WIND PRESSURES SITED ON BUILDINGS IN THE ULTIMATE 175 MPH WIND ZONE (NOMINAL WIND SPEED 136 MPH) & NEC.
2) THE OWNER/CONTRACTOR SHALL VERIFY ALL PRODUCT AVAILABILITY, DIMENSIONS, SITE CONDITIONS, AND EQUIPMENT REQUIREMENTS BEFORE COMMENCING ANY WORK. ALL DIMENSIONS AND CONDITIONS MUST BE VERIFIED IN THE FIELD. ANY DISCREPANCIES AND OMISSIONS SHALL BE BROUGHT TO THE ATTENTION OF THE ENGINEER BEFORE PROCEEDING WITH THE AFFECTED PART OF THE WORK.
3) THE STRUCTURE IS DESIGNED TO BE SELF SUPPORTING AND STABLE AFTER THE BUILDING IS COMPLETE. IT IS THE CONTRACTORS RESPONSIBILITY TO DETERMINE ERECTION PROCEDURES AND SEQUENCE TO ENSURE SAFETY OF THE BUILDING AND ITS COMPONENTS DURING ERECTION. THIS INCLUDES THE ADDITION OF NECESSARY SHORING, SHEETING, TEMPORARY BRACING, GUYS, OR TIE-DOWNS.
4) DESIGN LOADS
   THE STRUCTURAL SYSTEM FOR THIS BUILDING HAS BEEN DESIGNED IN ACCORDANCE WITH SECTION 1609 OF THE 2020 FLORIDA BUILDING CODE.
5) N/A
6) ALL SITE RELATED WORK SUCH AS, BUT NOT LIMITED TO, FOUNDATION, TIE DOWN AND ELECTRICAL SERVICE SHALL BE BY OTHERS AND AS PER THE AHJ. NO ELECTRICAL, PLUMBING OR HVAC WORK IS INCLUDED IN THIS SHELL DESIGN
7) SITE ENVIRONMENTAL STUDIES, IF REQUIRED, ARE TO BE PERFORMED BY OTHERS.
8) PRODUCT/MATERIAL SUBSTITUTION IS PERMITTED IF THE SUBSTITUTE IS EQUAL OR GREATER THAN THE SPECIFIED PRODUCT. TESTING DATA AND/OR VERIFICATION IS THE RESPONSIBILITY OF THE CONTRACTOR.
9) ALL REQUIRED PRODUCTS SHALL MEET FLORIDA PRODUCT APPROVAL RULE 61G20-3.006 (FAC)
10) ALL DIMENSIONS AND CONDITIONS MUST BE VERIFIED IN THE FIELD. DO NOT SCALE THE DRAWINGS. FOLLOW WRITTEN DIMENSIONS ONLY. ANY DISCREPANCIES SHALL BE BROUGHT TO THE ATTENTION OF THE ENGINEER PRIOR TO PROCEEDING WITH THE AFFECTED PART OF THE WORK.
11) CH 633 PLAN REVIEW AND INSPECTIONS SHALL BE CONDUCTED BY LOCAL FIRE AND SAFETY INSPECTOR

Project: Shed Ranch

Basic Building Structural Information

This table was prepared using KiloNewton LLC's internal calculations from ASCE 7-16

This information was created in accordance with Chapter 16 of the 2020 Florida Building Code. The Component and Cladding Pressures were generated using the method in Part 2 of Chapter 30 of ASCE 7-16.

Floor & Roof Live Loads	(R-3 • Single-Family Dwellings)
Attics:	20 psf w/ storage, 10 psf w/o storage
Habitable Attics, Bedroom:	30 psf
All Other Rooms:	40 psf
Garage:	40 psf
Roofs:	20 psf

(Balcony and Deck live loads are 150% of the adjacent space served.)

Wind Design Data

Ultimate Wind Speed: 175 mph Nominal Wind Speed: 136 mph
Risk Category: II Wind Exposure: B
Enclosure Classification: Enclosed End Zone Width (a): 4.00 ft.
Internal Pressure Coefficient: 0.18 Roof Geometry: Gable
Roof Slope: 5.0 in 12 (22.6°) Mean Roof Height: 20 ft.

(The Ultimate Wind speed was used to determine the Component and Cladding design pressures.)

(This Building is in a Wind-Borne Debris Region, and all exterior glazed openings shall be protected from wind-borne debris.)

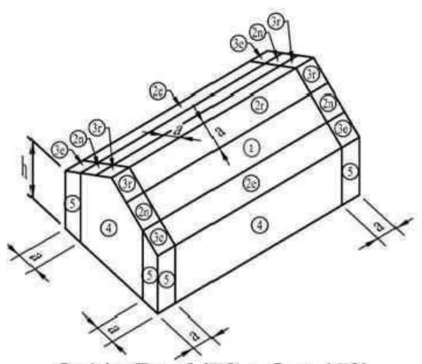
Components and Cladding	
Roof Zone 1:	+29.7 psf max., -61.58 psf min.
Roof Zone 2e:	+29.7 psf max., -61.58 psf min.
Roof Zone 2n:	+29.7 psf max., -108.22 psf min.
Roof Zone 2r:	+29.7 psf max., -108.22 psf min.
Roof Zone 3e:	+29.7 psf max., -108.22 psf min.
Roof Zone 3r:	+29.7 psf max., -122.22 psf min.
Wall Zone 4:	+38.25 psf max., -59.7 psf min.
Wall Zone 5:	+55.04 psf max., -73.7 psf min.

Design Soil Bearing Capacity: 2,000 psf

DRAWING INDEX

S.100	General Notes/Cladding/Wind Loads
S.200	Framing Plans/Anchoring
S.300-A	Building Foundation Plans & Details
S.300-B	Floor/Electrical Plans & Building Sections
S.400	Roof & Awning Plans/Details
A.100	Exterior Elevations

Components & Cladding



Gable Roof (7° < Θ ≤ 45°)

Notes for Future Conversion of Shell Shed to Single Family Residence:

1) THIS SHED / SHELL STRUCTURE HAS BEEN ENGINEERED TO MEET THE STRUCTURAL REQUIREMENTS FOR HUMAN OCCUPATION AS A SINGLE FAMILY RESIDENCE IN COMPLIANCE WITH FBC 2020 RESIDENTIAL WITH LATEST AMENDMENTS.
2) TO BE PREPARED FOR CONVERSION, SHELL MUST ALSO:
3) BE CONSTRUCTED WITH DOORS AND WINDOWS TO MEET MINIMUM EGRESS REQUIREMENTS FOR THE FINAL INTERIOR LAYOUT - BY OTHERS.
4) BE CONSTRUCTED WITH APPROVED VAPOR BARRIER BEHIND THE WALL SHEATHING
5) PROVIDE ADEQUATE DEPTH IN CEILING FOR REQUIRED INSULATION
6) ALL WORK DONE BEYOND SHELL STAGE IS TO BE DONE BY OTHERS UNDER SEPARATE PERMIT.

Code Notes

Code Version	2020 FBC Residential, 7th Ed. w/ 2021 Suppliments,
Electrical code	NEC - 2017
Building Type	STORAGE, MEETS R3 STANDARDS
Manufacturer	Florida Gulf Sheds, Inc
Agency Plan #	Florida-17
Construction Type	VB
Fire Protection	N/A
Fire Suppression System	N/A
Occupancy	UTILITY
Allowable Stories	1
Wind Velocity	175 mph* (see Material under Building Notes)
Fire Rating Exterior Walls	N/A
Max Floor Load	Live 120 psf, Dead 10 psf
Roof Load	Live 20 psf, Dead 8 psf
"R" Ratings	N/A
Modules per Building	1
Square Footage	Max 672 sqft
HVHZ Compliant	Design is not HVHZ Compliant

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Mary C. Alford, PE
Florida Professional Engineer #68187

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Shangri-la Building Plans

Florida Gulf Sheds, Inc.
2490 NE 200th Ave
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KILONEWTON

Date	Revision

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Albuquerque, NM 87102, USA
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Date: October 05, 2022

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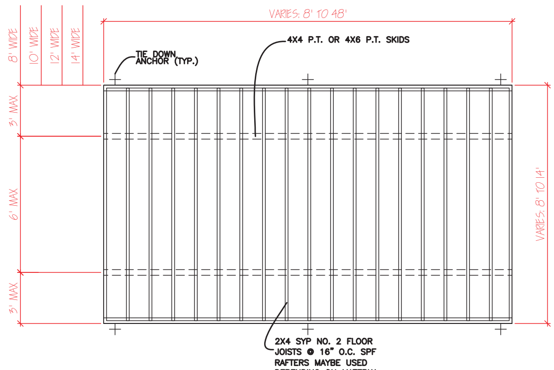
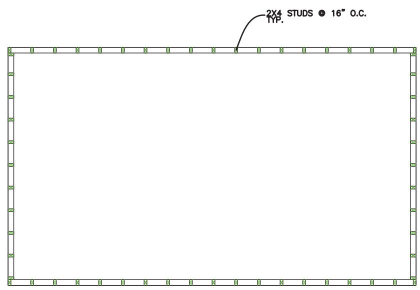
Drawing no: S.100



Floor Framing Plan 1/4" = 1'-0"



Wall Framing Plan 1/4" = 1'-0"

Header Schedule (Bearing Wall)

1) SPANS UP TO 4'–0": (2) 2X4; USE SPF NO. 2 UNLESS STATED OTHERWISE ON PLANS
2) SPANS UP TO 6'–0": (2) 2X6; USE SPF NO. 2
3) SPANS UP TO 8'–0": (2) 2X8; SPF NO. 2
4) SPANS UP TO 8'–0" @ END WALL ONLY: (2) 2x6 SYP NO. 2

NOTE: ALL BEAMS SHALL BE BUILT UP WITH 1/2" PLYWOOD OR 7/16" OSB FILLER. NAILING SHALL BE 16D NAILS @ 16" O.C. ALONG EACH EDGE. SPLICES, IF NECESSARY, SHALL BE LOCATED @ 1/4 THE LENGTH OF THE BEAM BETWEEN SUPPORTS.

Header Stud Requirements

MAXIMUM 8'–0" WALL HEIGHT

MAXIMUM HEADER SPAN (FEET):

1) 3'–0" (1) FULL LENGTH STUD. ATTACH HEADER TO STUDS WITH (1) SIMPSON FC4 FRAMING CLIP, OR USE (1) HEADER STUD & (1) FULL LENGTH STUD EACH SIDE.
2) 8'–0" (1) HEADER STUD, (1) FULL LENGTH STUD EACH SIDE.
3) USE (1) LSTA 12 HEADER TO STUD EACH SIDE.

Skid Placement Notes

1) FOR 14' & 12' WIDE BUILDINGS, PLACE THE EXTRA SKIDS 1FT FROM EACH WALL.

Tie-Down Anchors

1) PLACE WITHIN 9" EACH CORNER OF BUILDING (MIN. 4 REQUIRED).
2) SPACE NOT MORE THAN 8' O.C.; RUN PARALLEL TO THE RUNNERS.
3) TIE DOWN ANCHORS TO BE 1/2" DIA X 30" ROD, 4" SINGLE HELIX AUGER ANCHOR MODEL 59055, MANUFACTURED BY TIE–DOWN ENGINEERING, OR EQ.
4) INSTALL PER TIE–DOWN MANUFACTURER'S ENGINEERING SPECIFICATIONS.
5) WHERE APPLICABLE, INSTALLER MAY USE SINGLE OR STACKED, NON–GROUTED CMU BLOCK TO SUPPORT RUNNERS. BLOCKS SHALL BE PLACED ON 16"X16" ABS PADS AND SPACED MAXIMUM AS PER THE SCHEDULE BELOW ALONG THE LENGTH OF THE RUNNER. PT SHIM WOOD WEDGE MAY BE USED FOR LEVELING. EXPOSED OPEN CMU CELLS ARE ACCEPTABLE FOR SHED USE
6) THE TIE DOWN ANCHORS MUST MEET REQUIREMENTS OF APPLICABLE LOADS AND CODES

SPAN	4X4	4X6
14'	4'-6"	6'-0"
12'	5'-0"	7'-4"
10'	6'-0"	8'-0"
8'	6'-0"	8'-0"

Framing Notes

1) UNLESS STATED OTHERWISE ON PLANS, ALL FRAMING FOR 8'–0" HIGH WALLS SHALL BE 2X4 SPF #2 GRADE LUMBER UP TO 155MPH WIND SPEEDS AND FOR SPEEDS ABOVE 155MPH 2X4 SYP #2 GRADE LUMBER IS REQUIRED. STUDS SHALL BE SPACED AT 16" O.C.
2) ALL FRAMING COMING IN CONTACT WITH CONCRETE, EARTH, OR MASONRY SHALL BE PRESSURE TREATED.
3) DOUBLE BEARING TOP PLATES SHALL HAVE ALL JOINTS LAP–SPLICED. WITHIN THE CENTER THIRD OF A WALL LENGTH, THE MINIMUM LAP SHALL BE 4'–0". A 2'–0" MINIMUM LAP SPLICE IS REQUIRED ELSEWHERE. NAILING REQUIREMENT SHALL BE (16) 16D NAILS FOR CENTER THIRD OF WALL AND (12) 16D NAILS FOR OTHER APPLICATIONS. MINIMUM (1) STUD AT SPLICE LOCATION.
4) EXTERIOR SHEATHING SHALL BE EITHER:
SMARTSIDE ATTACHED WITH 8D NAILS SPACED @ 4" O.C. EDGE AND 6" INTERIOR
INNER–SEAL OSB APA RATED SHEATHING 7/16" OR 15/32". SHALL BE ATTACHED WITH 6D NAILS, 4" O.C. EDGE AND 6" O.C. INTERIOR. 15/32" SHALL BE ATTACHED WITH 8D NAILS, 4" O.C. EDGE AND 6" O.C. INTERIOR.
PNEUMATIC NAILS MAY BE USED IN LIEU OF 8D COMMONS, USE PASLODE 2– 3/8"x0.113 GALVANIZED OR EQUIVALENT.

Fastener Schedule for Structural Members

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING OF FASTENERS
JOIST TO SILL OR GIRDE, TOE NAIL	(3) 8D	--
1"X6" SUBFLOOR OR LESS TO EACH JOIST, FACE NAIL	(2) 8D	--
SOLE PLATE TO JOIST OR BLOCKING, FACE NAIL	16D	16" O.C.
TOP OR SOLE PLATE TO STUD, END NAIL	(2) 16D	--
STUD TO SOLE PLATE, TOE NAIL	(4) 8D TOE NAIL OR (2) 16D END NAIL	--
DOUBLE STUDS, FACE NAIL	16D OR 3"X0.131 @ 8" O.C.	24" O.C.
DOUBLE TOP PLATES, FACE NAIL	(8) 16D NAILS	LAP SPLICE
	16D	16" O.C.
SOLE PLATE TO JOIST OR BLOCKING AT BRACED WALL PANELS	(3) 16D	16" O.C.
DOUBLE TOP PLATES, MINIMUM 24 INCH OFFSET OF END JOINTS, FACE NAIL IN LAPPED AREA	(8) 16D	--
BLOCKING BETWEEN JOISTS OR RAFTERS TO TOP PLATE, TOE NAIL	(3) 8D	--
RIM JOIST TO TOP PLATE, TOE NAIL	8D	6" O.C.
TOP PLATES, LAPS AT CORNERS AND INTERSECTIONS, FACE NAIL	(2) 16D	--
BUILT–UP HEADER, TWO PIECES WITH ½" SPACER	16D	16" O.C. ALONG EACH EDGE
CONTINUED HEADER, TWO PIECES	16D	16" O.C. ALONG EACH EDGE
CEILING JOISTS TO PLATE, TOE NAIL	(3) 8D	--
CONTINUOUS HEADER TO STUD, TOE NAIL	(4) 8D	--
CEILING JOIST, LAPS OVER PARTITIONS, FACE NAIL	(3) 16D	--
CEILING JOIST TO PARALLEL RAFTERS, FACE NAIL	(3) 16D	--
RAFTER TO PLATE, TOE NAIL	(3) 10D	--
1" BRACE TO EACH STUD AND PLATE, FACE NAIL	(2) 8D	--
BUILT–UP CORNER STUDS	16D	24" O.C.
BUILT–UP GIRDERS AND BEAMS	20D @ 32" O.C. OR 3"X0.131 @ 24" O.C.	NAIL EACH LAYER AS FOLLOWS: STAGGER @ TOP AND BOTTOM. TWO NAILS @ EACH END AND @ SPLICE
ROOF RAFTERS TO RIDGE, VALLEY OR HIP RAFTERS: TOE NAIL FACE NAIL	(4) 16D (4) 16D	-- --
BUILT–UP LVL BEAM – TOP LOAD 2 PLY, MAX. DEPTH 12"	16D	2 ROWS, 12" O.C. STAGGERED TOP & BOTTOM
DEPTH > 12"	16D	3 ROWS, 12" O.C. STAGGERED TOP & BOTTOM
4 OR MORE PLY	½" BOLTS	½" A–307 BOLTS TOP & BOTTOM, STAGGERED 24" O.C. KEEP 3" FROM EDGES

NOTE:
FOR ADDITIONAL NAILING REQUIREMENTS, REFER TO SECTION 2304.9, FASTENING AND TABLE 2304.9.1, FASTENING SCHEDULE IN THE FLORIDA BUILDING CODE 2017, 6TH EDITION.

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Florida Gulf Sheds, Inc.
2490 NE 200th Ave
Willston, FL 32696

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505 312-8490

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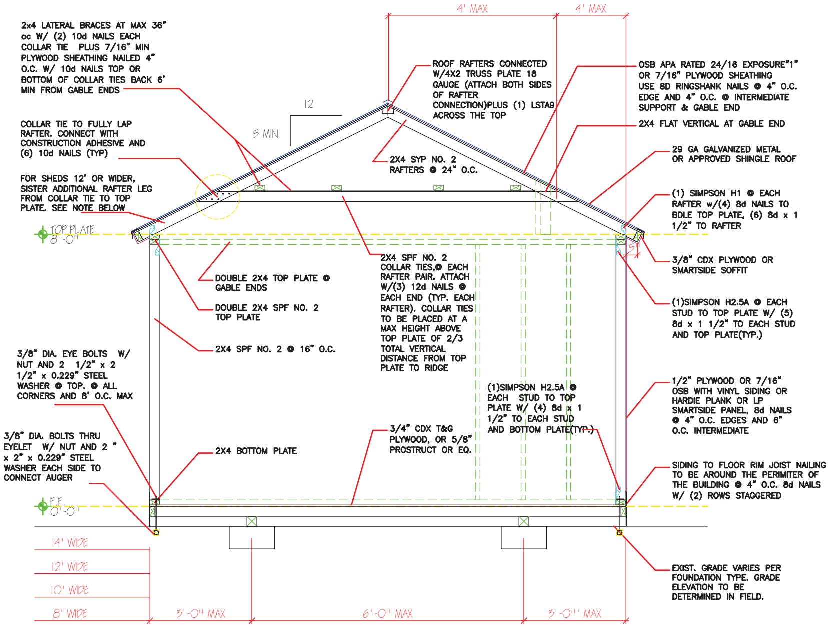


Typ. Building Section @ Gable 1/2" = 1'-0"

MIN 4x4 PT SKIDS TO BE PLACED AT MAXIMUM SPACING AS ABOVE. (2) SKIDS MIN. UP TO 12' WIDTH. USE (3) OR (4) SKIDS FOR LARGER WIDTHS

—OR —

USE 2x6 PT FLOOR JOISTS @ 16" oc UP TO 8' FREE SPAN

FOR SHED 12' WIDE AND OVER, USE 2x6 RAFTERS

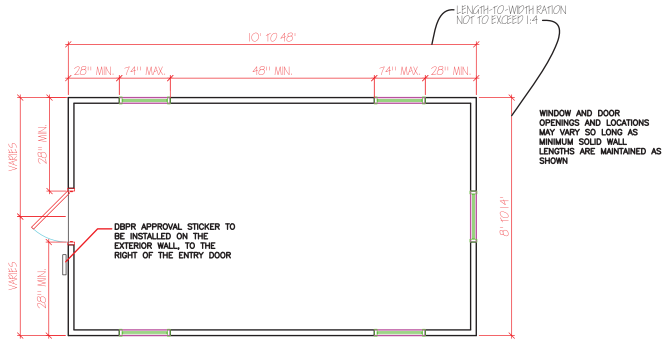
—OR —

SISTER ADDITIONAL 2x4 RAFTER LEG FROM DIRECTLY BELOW COLLAR TIE TOP TOP PLATE

ENDS OF COLLAR TIES TO BE PLACED NO MORE THAN 4' (MEASURED HORIZONTALLY) FROM EITHER END OF RAFTER MEMBER.

ALL WOOD PLACE WITHIN 12" OF GRADE SHALL BE TREATED FOR TERMITE RESISTANCE.

GUTTERS MAY BE INSTALLED ON SITE BY OTHERS.



Typical Floor Plan 1/4" = 1'-0"

DOOR AND WINDOW SCHEDULE

CATEGORY	SUB CATEGORY	MANUFACTURER	PRODUCT DESCRIPTION	APPROVAL NUMBER
		Doors		
EXTERIOR DOORS	SWINGING	Jeld-Wen	Steel Nine-light	FL 12769.2
		Windows		
WINDOWS	SINGLE / DOUBLE HUNG	Jeld-Wen	Single Hung	FL 14095.5
		MISC		
ROOFING PRODUCT	NON-STRUCT METAL	Reeds Metal	CERAM-A-STAR	FL 24508 R1
PANEL WALL	SIDING	LP	Smartside	FL 9190.1
PANEL WALL	SOFFITS	LP	Smartside	FL 9103.1

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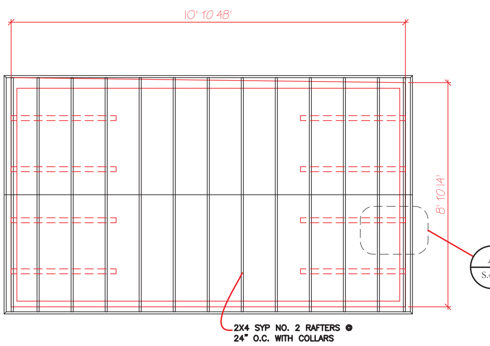
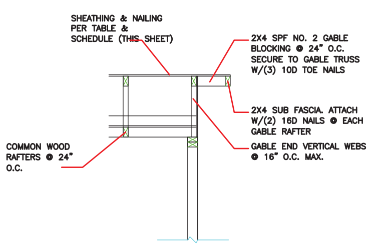


Roof Framing Plan 1/4" = 1'-0"



A - Drop Gable, 6" to 12" Overhang 1/2" = 1'-0"

Roof Sheathing Fastening Notes

NAILING:

ALL ZONES: USE 8D RINGSHANK NAILS 4" O.C. EDGE & 4" INTERMEDIATE

NOTE:

IF PNEUMATIC NAILS ARE USED, USE A PASLODE OR EQUIVALENT, 2-3/8"X0.113 THREADED, COATED NAIL IN LIEU OF 8D RINGSHANK NAILS CALLED FOR ABOVE.

Roof Coverage

29 GA GALVANIZED METAL ROOF OR SHINGLES SHALL COMPLY WITH ASTM D 7158 CLASS H OR ASTM D 3161 CLASS F OR TAS 107 OR AND R905.2.6.1 & TABLE R905.2.6.1 . ROOFING SHALL HAVE FLORIDA OR MIAMI - DADE PRODUCT APPROVAL FOR ULT WIND SPEED OF 180 MPH. INSTALL PER MANUFACTURER'S INSTRUCTIONS

Sheathing Requirements

SHINGLE OR METAL ROOF:
7/16" OR 1/2" C-D, GROUP 2, EXP. 1 APA RATED 24/16

WHERE PERMITTED BY LOCAL AUTHORITY, INNER SEAL OSB SHEATHING CAN BE USED:
SHINGLE OR METAL ROOF:
7/16", 5/32", OR 1/2" APA RATED 24/16

NOTES:

1) PLYWOOD TO BE PERPENDICULAR TO FRAMING. END JOINTS SHALL BE STAGGERED.
2) CONTRACTOR SHALL INSTALL PLYWOOD USING PLYWOOD CLIPS WITH BUILT-IN SPACERS.
3) UNDERLAYMENT: REFERENCE TABLE R905.1.1 FOR UA=UNDERLAYMENT ATTACHMENT

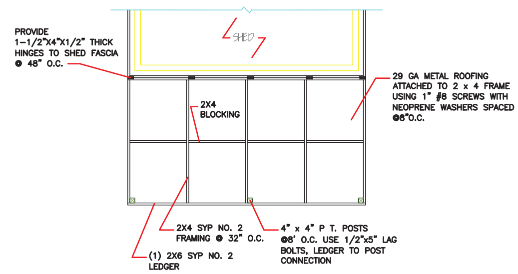
   ASPHALT SHINGLES/METAL ROOF

   4:12 AND GREATER
   ASTM D 226 TYPE 11, UA2
   ASTM D 4869 TYPE IV, UA2 ASTM D 6757, UA2
   ASTM D 1970, UA3
4) ALL MEMBRANE FLASHINGS INSTALLED PER MANUFACTURER'S SPECIFICATIONS

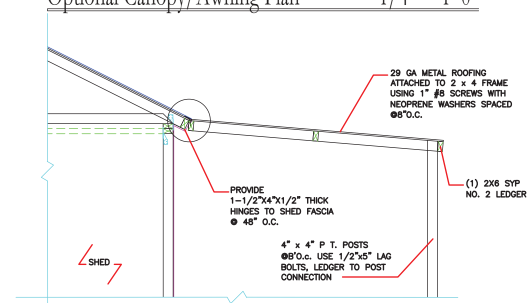
THE ABOVE BUILDING/STRUCTURE HAS BEEN DESIGNED IN ACCORDANCE WITH ASCE 7-16 FOR GRAVITY ANO DESIGN PRESSURES GENERATED BY A ULTIMATE WIND SPEED OF 175 M.P.H . 3 SECOND GUST & NOMINAL WIND SPEED OF 136 M.P.H., 3 SECOND GUST.

Awning Notes

AWNING IS OPTIONAL. IF AWNING IS TO BE ADDED, IT SHALL BE IN COMPLIANCE WITH ADDITIONAL CALCULATIONS SUPPLIED BY OUTSIDE ENGINEER.



Optional Canopy/Awning Plan 1/4" = 1'-0"



Optional Canopy/Awning Plan 1/4" = 1'-0"

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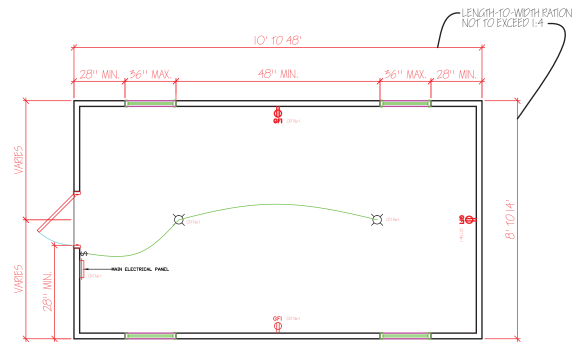
Drawing no:

S.400



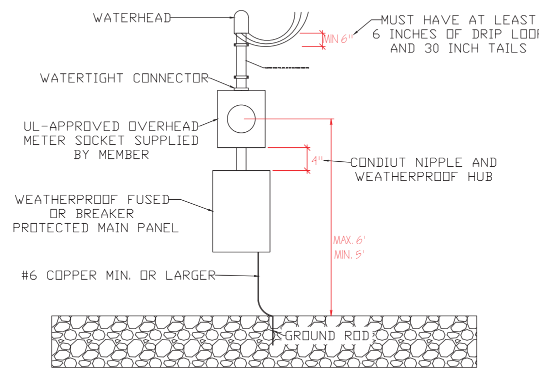
Typ. Floor/Electrical Plan 1/4" = 1'-0"

ELECTRICAL NOTES

1) THE OUTLET LOCATIONS (only) CAN BE ADJUSTED AS NEEDED
2) ALL THE OUTLETS ARE GFCI PROTECTED.
3) THE OUTLETS AND LIGHTS CONFIGURATION IS FOR A "STANDARD PACKAGE". ANY ADDITIONAL ELECTRICAL COMPONENTS INSTALLED NEED TO BE EVALUATED AS NEEDED.
4) ALL ELECTRIC WORK TO BE IN ACCORDANCE WITH NEC 2017.
5) ALL THE CIRCUITS WIRES ARE 14 GAUGE. ROMEX SIMpull 14/2 SOLID INDOOR NON-METALLIC WIRE OR EQUIVALENT CAN BE USED.
6) A SINGLE 15 AMP CIRCUIT USED FOR ALL FIXTURES.
7) ALL NM, NMC, and NMS SHALL BE INSTALLED IN ACCORDANCE WITH NEC SECTION 300 AND 334.
Nm, NMC and NMS SHALL ONLY BE USED ON STRUCTURES THAT ARE ACCESSORY USE TO RESIDENTIAL ONE AND TWO FAMILY STORAGE STRUCTURES PER SECTION 334.10(1).
8) ELECTRICAL OUTLETS SHALL BE SPACED MAXIMUM OF 12' OC FOR ALL SHED CONFIGURATIONS.
9) A MIN. OF 60 AMP MAIN ELECTRICAL PANEL SHOULD BE INSTALLED. 100AMP 6-SPACES 12-CIRCUIT INDOOR CONVERTIBLE MAIN LUG LOAD CENTER OR EQUIVALENT CAN BE USED

ELECTRICAL SYMBOL LEGEND

	CEILING LIGHT
GFI	OUTLET GROUND FAULT INTERRUPTER
$	SWITCH SINGLE POLE
	BREAKER BOX with 15Amp Capacity



METER LOOP DETAIL NTS

ELECTRICAL PANEL SCHEDULE

CKT. No.	AMP RATING	CONNECTED AMPS	CONNECTED LOAD WATTS	DESCRIPTION	BRANCH CKT FACTOR	FEEDER FACTOR	BRANCH CKT. LOAD	QUANTITY	TOTAL FEEDER LOAD
1	15 - SP	3.00	180.00	Recepts	1.00	1.00	360.00	3	540
1	15 - SP	3.54	340.00	Lights	1.25	1.25	425.00	0	0
NEC CALCULATED LIGHT LOAD IF GREATER THAN ACTUAL LOAD =									2016.00
CALCULATED LIGHTING LOAD EXCEEDS CONNECTED LIGHTING LOAD									
NEC RECEPTS FIRST 10,000 @ 100% AND ALL OTHERS @ 50% =									0
TOTAL WATTS									2556.00

$\frac{2556.00}{240} = 10.65$ AMPS

FLORIDA - 20

Mary C Alford

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Mary C. Alford, PE
Florida Professional Engineer #68187

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Seal & Sign

Shangri-la Building Plans

Florida Gulf Sheds, Inc.
2490 NE 200th Ave
Willston, FL 32696

KILONEWTON

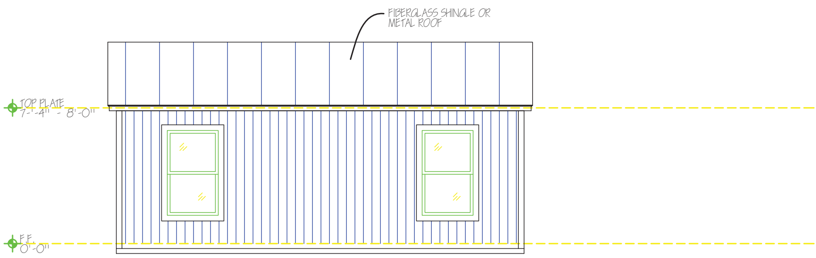
Date	Revision

610 Gold AVE SW, Suite 236
Albuquerque, NM 87102, USA
505 312-8490

Date: October 05, 2022

www.kilonewtonllc.com

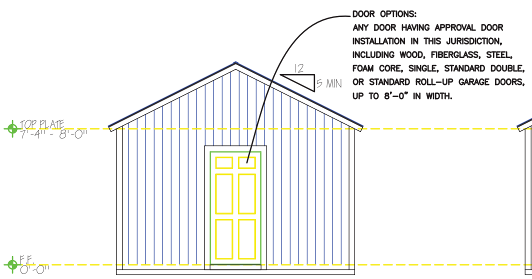
Drawing no: E.100



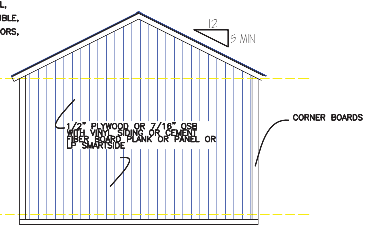
Side Elevation (Typ.) 1/4" = 1'-0"

Elevation Notes

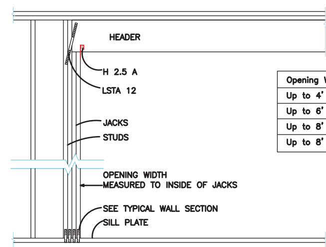
1) DOOR AND WINDOW QUANTITY/LOCATION VARIES
2) IF STRUCTURE IS PLACED ON CONCRETE SLAB & FOUNDATION. STOP SIDING AT FINISHED FLOOR.



Front Elevation 1/4" = 1'-0"



Rear Elevation 1/4" = 1'-0"



Opening Width	# of Jacks	# of Studs	Min Header Size
Up to 4'	1	1	(2) 2x4
Up to 6'	1	2	(2) 2x6
Up to 8'	2	2	(2) 2x8
Up to 8' @ End Wall	2	2	(2) 2x6

Notes:

a) Based on Uniform Loads. Heavy Concentrated Loads require Engineering Review.
b) Number of Jack & Stud Requirements per Opening Width 2x4 or 2x6 SPF #1&2 Construction – Max Wall height = 12' (Based on 16" O.C. Stud spacing).
c) Equal to Simpson ties noted are acceptable.

Typical Opening Details NTS

*****NOTE*****
The base structure is designed for use as a non-occupied storage building. If this building is being prepared for possible conversion to Residential Occupancy by Others: An approved vapor barrier shall be included in the wall section, door and window sizes shall meet egress requirements where needed, and all electrical circuits shall be Arc Fault / GFI where required per FBC Residential. It is the Owners' responsibility to provide additional full permit documents, scope of work and application meeting all FBC Residential requirements to local jurisdiction prior to any residential improvements or occupancy.

FLORIDA - 20

Mary C Alford
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Date: 2023.02.10 12:36:02 -05'00'

Mary C. Alford, PE
Florida Professional Engineer #68187

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Seal & Sign

Shangri-la Building Plans

Florida Gulf Sheds, Inc.
2490 NE 200th Ave
Willston, FL 32696

KILONEWTON

Date	Revision

610 Gold AVE SW, Suite 236
Albuquerque, NM 87102, USA
505 312-8490

Date: October 05, 2022

www.kilonewtonllc.com

Drawing no: A.100

SEPTEMBER 9, 2022

KILONEWTON

Mary C Alford Digitally signed by Mary C Alford Date: 2022.09.09 07:12:15 -04'00'	Mary C Alford PE 13880 SW 173rd Court Archer, FL 32618
Mary C Alford, State of Florida Professional Engineer, License #68187	

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FLORIDA GULF SHEDS STRUCTURAL ANALYSIS & DESIGN

VERSION 1

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Revision History

Version	Date	Description	Reviewed By
1	08/17/2022		John Williamson
2	08/18/2022		Mary Alford
3	09/07/2022		John Williamson
4	09/08/2022		Mary Alford

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s

1 Executive Summary

KiloNewton has performed the structural analysis and design of Florida Gulf Sheds based on the Florida Building Code (FBC) 2020 and International Building Code (IBC) as needed. The wind speeds and velocity pressure calculations were performed based on ASCE 7-17 Chapters 26-30 as needed. The material properties and required strength checks for various components were obtained from National Design Specification (NDS) for wood construction. **Figure 1** provides the wind speed zones for the State of Florida. The current shed design can be implemented in parts of Florida with wind speeds reaching up to 175mph. SPF studs can be used up to 155mph and SYP studs are needed for speeds above 155mph up to 175mph.

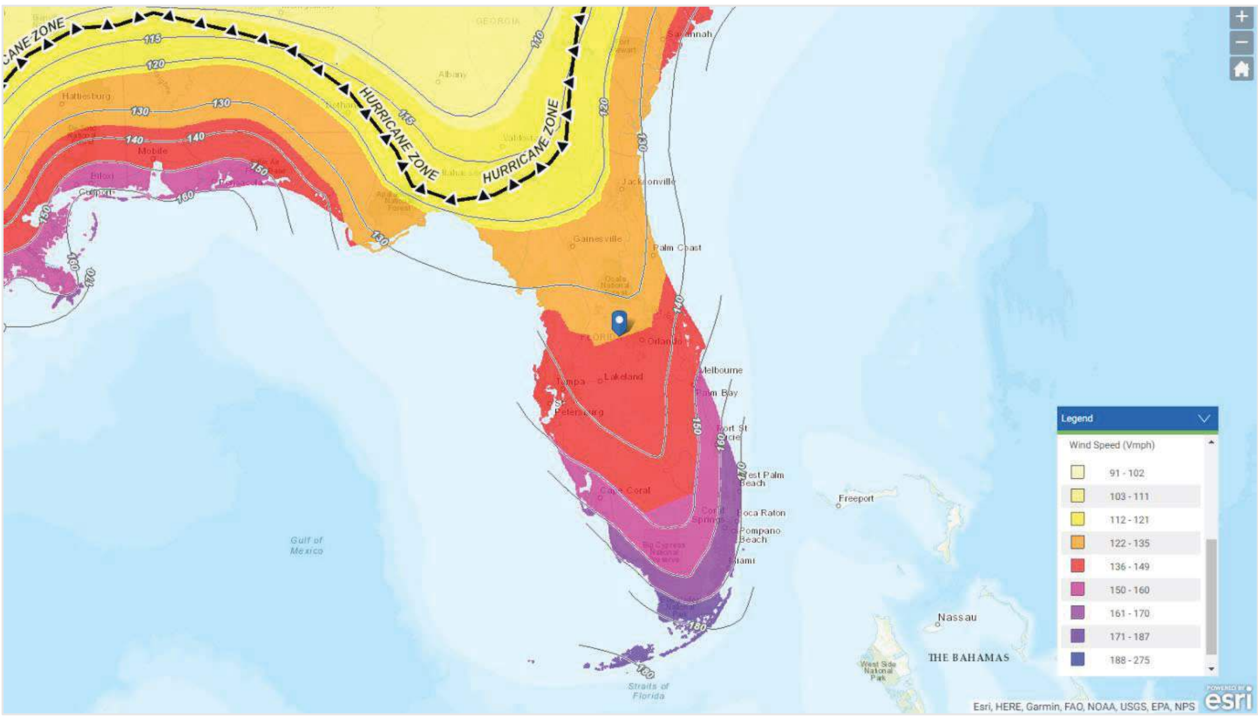

Figure 1: Wind Speed Map for Florida

2 Introduction

The structural analysis of different shed configurations was performed for wind speeds prevalent in the State of Florida. The structural specifications such as member sizes, panel sizes, perforations (i.e., doors and windows), and member spacings among others were obtained from the CAD file shared with KiloNewton along with multiple communications with the client. Any changes to the original understanding (i.e., details from PDF drawing Titled: *Master Shed Plan*, dated – *1/29/2018*) of the shed layout and materials and member sizes were updated on the CAD file as needed. This analysis is for the SHELL ONLY and foundations and attachments must be in compliance with the local AHJ's requirements.

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3 Loads

3.1 Dead Loads and Live Loads

The dead load and live loads are provided in the CAD file shared with KiloNewton. These loads are used in analyzing the structural members of the shed. The dead load and live load for floor are *10psf* and *120psf* respectively. The dead load and live load for the roof are *8psf* and *20psf* respectively.

3.2 Wind Loads

ASCE 7-16 Chapter 26 was used to determine the building type and wind pressures, Chapter -27, 28, and 30 were used to determine the surface loads on wall and gable roofs for all zones. Based on section 26.2 definitions, the building is classified as an Enclosed building during the critical wind event.

The set of equations provided in **Table 1** are used to estimate the velocity pressures and design wind pressures. A sample of velocity pressure calculation is provided in **Table 2** for 175mph.

Table 1: Load calculation equations

Quantity	Equation	Reference
Velocity Pressure, q_h, q_z,	$q_h = 0.00256 K_z K_{zt} K_d K_e V^2$	ASCE 7-16: Eq. 26.10-1
Design Wind Pressure, *p*	$p = q_h[(GC_p) - (GC_{pi})]$	ASCE 7-16: Eq. 30.3-1
Parameters		
Velocity pressure exposure coefficient	K_z	ASCE 7-16: Section:26.10.2
Topographic factor	K_{zt}	
Wind directionality factor	K_d	
Ground elevation factor	K_e	
External pressure coefficients	GC_p	ASCE 7-16: Section:30.3.2
Internal pressure coefficients	GC_{pi}	

Table 2: Sample Calculations for velocity pressure

Velocity Pressure		
Category	I	
Wind Speed, V	175	mph
K_z	0.7	Exposure-B
K_{zt}	1	
K_d	0.85	
K_e	1	
Velocity Pressure (qz = qh)	46.65	psf

The external wall pressure, GC_p values for walls were obtained using ASCE 7-16 Fig. 30.3-1 provided in **Figure 2**. The internal wall pressure values, GC_{pi} were obtained from ASCE 7-16 Table 26.13-1 provided in **Figure 3**. The roof pressures are estimated from the ASCE 7-16 Fig-30.3-2C shown in **Figure 4**.

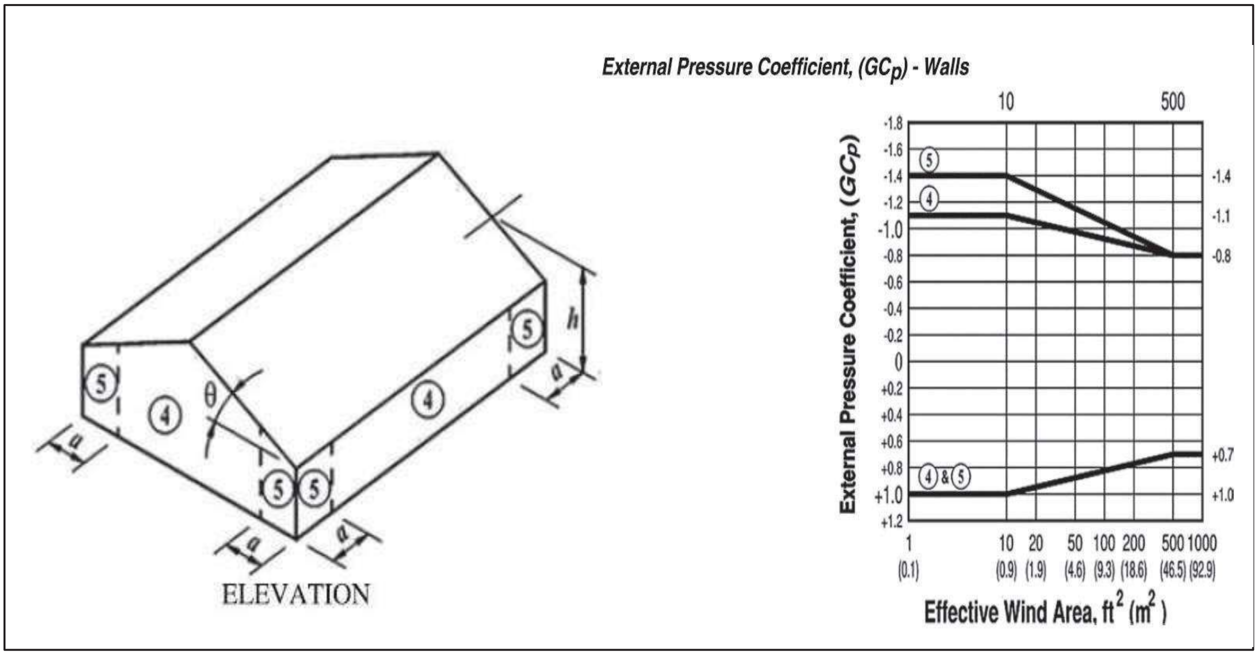


Figure 2: External Wall Pressure Zones and Coefficient

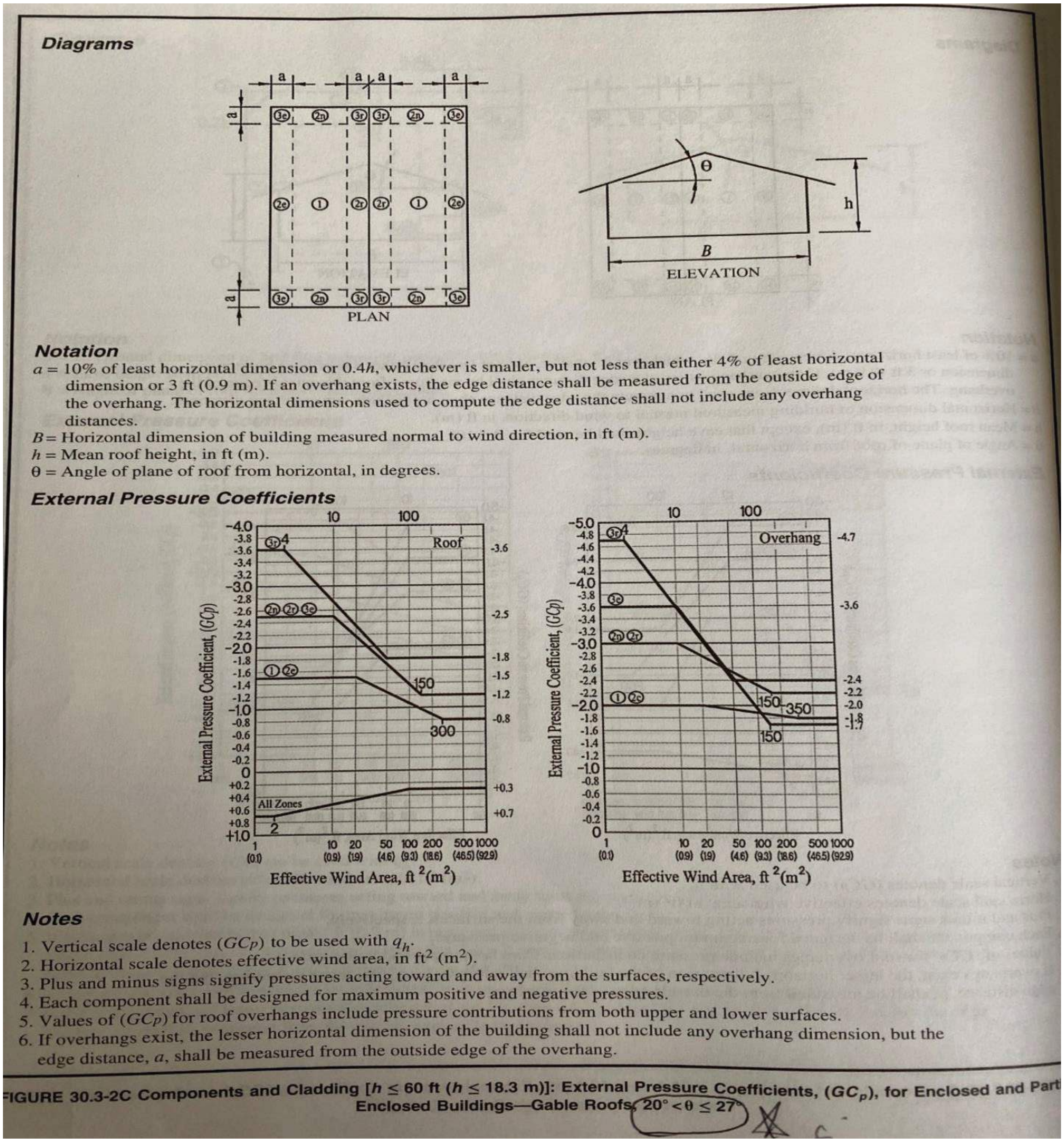
Table 26.13-1. Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, (GC_{pi}), for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof).

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 ft² (0.37 m²), and $A_{oi}/A_{gi} \leq 0.2$	Moderate	+0.18 −0.18
Partially enclosed buildings	$A_o > 1.1A_{oi}$, and $A_o >$ the lesser of $0.01A_g$ or 4 ft² (0.37 m²), and $A_{oi}/A_{gi} \leq 0.2$	High	+0.55 −0.55
Partially open buildings	A building that does not comply with Enclosed, Partially Enclosed, or Open classifications	Moderate	+0.18 −0.18
Open buildings	Each wall is at least 80% open	Negligible	0.00

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
   (a) A positive value of (GC_{pi}) applied to all internal surfaces, or
   (b) A negative value of (GC_{pi}) applied to all internal surfaces.

Figure 3: Internal wall pressure coefficients for different building types



Diagrams

Notation

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m). If an overhang exists, the edge distance shall be measured from the outside edge of the overhang. The horizontal dimensions used to compute the edge distance shall not include any overhang distances.

B = Horizontal dimension of building measured normal to wind direction, in ft (m).

h = Mean roof height, in ft (m).

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficients

Notes

1. Vertical scale denotes (GC_P) to be used with q_h.
2. Horizontal scale denotes effective wind area, in ft^2 (m^2).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GC_P) for roof overhangs include pressure contributions from both upper and lower surfaces.
6. If overhangs exist, the lesser horizontal dimension of the building shall not include any overhang dimension, but the edge distance, a, shall be measured from the outside edge of the overhang.

FIGURE 30.3-2C Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, (GC_p), for Enclosed and Part[ially] Enclosed Buildings—Gable Roofs, $20° < \theta \leq 27°$

Figure 4: Roof Pressures for Roof tilt 20-27 deg.

3.2.1 Wall Pressures

The external and internal pressure coefficients and pressures are shown in **Table 3** and **Table 4**. These pressures were identified based on the zones provided by ASCE:7-16 wall pressure diagram shown in **Figure 2**. Similarly, the zonal coefficients and pressures are estimated for roof zones are presented in **Table 5** and **Table 6** respectively. The positive and negative zones in **Table 3** show the pressures acting towards and away from the wall respectively. The GC_p and GC_{pi} are the external and internal pressure coefficients respectively. The coefficients were estimated based on the effective wind area defined by ASCE7-16 (i.e., larger value of span x spacing or $span^2/3$). The length and width of building sizes analyzed in this report are 8'x10', 10'x28', 12'x32', and 14'x40' with wall height of 8' and roof raise of 5" in 12".

Table 3: External and Internal Pressure Coefficients.

Zones	All Shed Sizes
Negative Zone # 5, GC_p	-1.4
Negative Zone # 4, GC_p	-1.1
Positive Zone #4 & #5, GC_p	1
Negative, Gc_{pi}	-0.18
Positive, Gc_{pi}	0.18

A sample wall pressures calculations is shown below where Zone#5 pressures are estimated based on two different combinations. Case-1: the wall experiencing negative pressures externally and internally and Case-2: the wall experiencing negative and positive pressures externally and internally. The worst-case loads are used for analyzing the shear wall requirements in the later sections of this report. **Table 4** shows the pressures on walls for all building sizes requested by the client.

Zone #5: P_w = wind pressure

- Case-1 – Internal Suction: Wind Pressure, $Pw = q * (GCp - GCpi)$

  $= 46.65 * (-1.4 — (-0.18))$

  $= -56.91\ psf$

- Case-2 – Internal Pressurization: Wind Pressure, $Pw = q * (GCp - GCpi)$

  $= 46.65 * (-1.4 — (0.18))$

  $= -73.70\ psf$

Table 4: Wall Pressures on All Shed Sizes

Wall Zones		Pw (psf)
Zone #5 case 1	Leeward Wall (Edge)	-56.911
Zone #5 case 2	Leeward Wall (Edge)	-73.704
Zone #4 case 1	Leeward Wall (Interior)	-42.916
Zone #4 case 2	Leeward Wall (Interior)	-59.709
Positive Zone #4 & #5 case 1	Windward Wall	55.045
Positive Zone #4 & #5 case 2	Windward Wall	38.251

3.2.2 Roof Pressures

The roof coefficients are obtained similar to wall coefficients as explained in Section **3.2.1**. using effective wind area. **Table 5** presents coefficients identified using ASCE plot presented in **Figure 4** for different building sizes.

Table 5: External Roof Pressure Coefficients

Zones	Bldg. 8'x10'	Bldg. 10'x28'	Bldg. 12'x32'	Bldg. 14'x40'
Zone 1	-1.5	-1.5	-1.5	-1.5
Zone 2e	-1.5	-1.5	-1.5	-1.5
Zone 2r	-2.5	-2.5	-2.4	-2.3
Zone 2n	-2.5	-2.5	-2.4	-2.3
Zone 3r	-2.8	-2.7	-2.65	-2.5
Zone 3e	-2.5	-2.5	-2.4	-2.3
All positive	0.58	0.56	0.55	0.5
Zone Minus, Gc_{pi}	-0.18	-0.18	-0.18	-0.18
Zone Plus, Gc_{pi}	0.18	0.18	0.18	0.18

The sample calculation shown below is similar to the one explained in Section 3.2.1 and Zone-1 example is presented for case-1 and case-2. The coefficients for all zones and cases and for all building sizes are presented in **Table 7**. The worst loading case between case-1 and case-2 were used for the analysis.

- Case-1: Wind Pressure, $Pw = q * (GCp - GCpi)$

  = $46.65 * (-1.5 — (-0.18))$

  = $-61.58 psf$

- Case-2 Wind Pressure, $Pw = q * (GCp - GCpi)$

  = $46.65 * (-1.5 — (0.18))$

  = $-78.37 psf$

Table 6: Roof Pressures estimated for different zones and the Shed sizes

Zones	B8x10 (psf)	B10x28 (psf)	B12x32 (psf)	B14x40 (psf)
Zone-1 case 1	-61.58	-61.58	-61.58	-61.58
Zone-1 case 2	-78.37	-78.37	-78.37	-78.37
Zone-2e case 1	-61.58	-61.58	-61.58	-61.58
Zone-2e case 2	-78.37	-78.37	-78.37	-78.37
Zone-2r case 1	-108.22	-108.22	-103.56	-98.89
Zone-2r case 2	-125.02	-125.02	-120.35	-115.69
Zone-2n case 1	-108.22	-108.22	-103.56	-98.89
Zone-2n case 2	-125.02	-125.02	-120.35	-115.69
Zone-3r case 1	-122.22	-117.55	-115.22	-108.22
Zone-3r case 2	-139.01	-134.35	-132.01	-125.02
Zone-3e case 1	-108.22	-108.22	-103.56	-98.89
Zone-3e case 2	-125.02	-125.02	-120.35	-115.69
All positive Zones case 1	35.45	34.52	34.05	31.72
All positive Zones case 2	18.66	17.73	17.26	14.93

4 Structural Analysis

4.1 Shear wall requirement

The shear wall length requirements were estimated using the wall pressures provided in **Table 4**. The shear capacity for structural sheathing estimated in **Table 7** for sheds is per FBC-2020 requirements. The allowable shear value provided in **Table 7** is for staples, however, it may be applied to common nails (i.e., 8d common nails) as long as the support provided remains the same. And IBC reference is provided in **Figure 5** which shows higher capacity (check nominal thickness 15/32”) for sheathing at 4” OC spacing. Therefore, the current calculations use the FBC values, conservatively.

Table 7: Shear wall capacity calculation for structural sheathing.

Allowance for wind design	1.4	Allowance permitted for wind design – FBC Section 2306.3
Allowable shear value, *plf*	280	Sheathing capacity 4” OC staples – FBC Table 2306.3(1)
Specific Gravity	0.42	Specific gravity of SYP and SPF
SG adjustment factor	0.82	FBC Table 2306(1): Note-“a”
Shear capacity, *plf*	321.44	$V_{allow} = 280x0.82x1.4$

Table 8 provides the shear wall length requirements for different shed sizes analyzed. Sample calculations for each column **Table 8** are provided below the table. The minimum shear wall “full height segments” provided in **Table 8** are per each wall. A full height segment is defined as the length of the shear wall without perforations/openings (i.e., windows and doors). FBC and IBC specify that the aspect ratio of each full-height wall segments should be (H/L) is **3.5:1** *minimum*. The wall height for all shed sizes is 8ft, therefore, the minimum spacing for placing an opening is **28 inches** (i.e., [8x12]/3.5 = 27.42”) on either side of the opening.

TABLE 1

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR APA PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR, LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b,h,i,j,k] (See also IBC Table 2306.4.1)

Panel Grade	Minimum Nominal Panel Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Panels Applied Direct to Framing: Nail Size (common or galvanized box)[k]	Nail Spacing at Panel Edges (in.): 6	4	3	2[e]	Panels Applied Over 1/2" or 5/8" Gypsum Sheathing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.): 6	4	3	2[e]
APA STRUCTURAL I grades	5/16	1-1/4	6d (0.113" dia.)	200	300	390	510	8d (0.131" dia.)	200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	230[d]	360[d]	460[d]	610[d]	10d (0.148" dia.)	280	430	550[f]	730
	7/16			255[d]	395[d]	505[d]	670[d]					
	15/32			280	430	550	730					
	15/32	1-1/2	10d (0.148" dia.)	340	510	665[f]	870		—	—	—	—
APA RATED SHEATHING; APA RATED SIDING[g] and other APA grades except Species Group 5	5/16 or 1/4[c]	1-1/4	6d (0.113" dia.)	180	270	350	450	8d (0.131 dia.)	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	220[d]	320[d]	410[d]	530[d]	10d (0.148" dia.)	260	380	490[f]	640
	7/16			240[d]	350[d]	450[d]	585[d]					
	15/32			260	380	490	640					
	15/32	1-1/2	10d (0.148" dia.)	310	460	600[f]	770	—	—	—	—	—
	19/32			340	510	665[f]	870	—	—	—	—	—
APA RATED SIDING[g] and other APA grades except Species Group 5			Nail Size (galvanized casing)					Nail Size (galvanized casing)				
	5/16[c]	1-1/4	6d (0.113" dia.)	140	210	275	360	8d (0.131" dia.)	140	210	275	360
	3/8	1-3/8	8d (0.131" dia.)	160	240	310	410	10d (0.148" dia.)	160	240	310[f]	410

Figure 5: IBC Structural Sheathing Allowable Shear Capacities

Table 8: Shear Wall Calculations and minimum shear wall length requirements for different shed sizes

Bldg. Width	Zone-4 area (ft^2)	Zone-5 area (ft^2)	Zone-4 load (lbs)	Zone-5 load (lbs)	Controlling shear load (lbs)	Load per wall (lbs)
8	19.92	51.75	-1189.41	-3814.17	-5003.59	-2501.79
10	36.58	51.75	-2184.47	-3814.17	-5998.64	-2999.32
12	54.25	51.75	-3239.36	-3814.17	-7053.53	-3526.77
14	73.58	51.75	-4393.78	-3814.17	-8207.95	-4103.98

Zone 4 and 5 areas:

These values are estimated per building based on the value – *'a'* defined in **Figure 2**.

Zone-4 loads:

For 8' wide bldg. $= 59.709 * 19.92$

$= -1189.41\ lbs$

Zone-5 loads:

For 8' wide bldg. $= 73.70 * 51.75$

$= -3814.17\ lbs$

Controlling load

Total force acting on the face of the wall = $(-1189.41\ lbs) + (-3814.17\ lbs)$

$$= -5003.59\ lbs$$

Load per wall

The total load is shared by two shear walls on either side and therefore, the shear load per wall

$$= -\frac{5003.59}{2} lbs = 2501.79\ lbs$$

4.2 Window/Openings requirements

Perforation or openings in the wall reduces it shear capacity of the wall and the reduced capacity can be estimated using a shear-reduction adjustment factor, C_o. The C_o values vary based on the wall height, maximum opening height, and percentage of "full-height" segments. The values of C_o are provided in **Figure 6**. Based on these values, the adjusted capacities for perforated shear walls were estimated in **Table 9**.

WALL HEIGHT (h)	MAXIMUM OPENING HEIGHT RATIO [a] AND HEIGHT				
	h/3	h/2	2h/3	5h/6	h
8'-0"	2'-8"	4'-0"	5'-4"	6'-8"	8'-0"
10'-0"	3'-4"	5'-0"	6'-8"	8'-4"	10'-0"
Percent Full-Height Sheathing [b]	**Shear Capacity Adjustment Factor**				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

a. The maximum opening height ratio is calculated by dividing the maximum opening clear height by the shearwall height, h. If areas above and below an opening remain unsheathed, the height of the opening shall be defined as the height of the wall.

b. The percent of full height sheathing is calculated as the sum of widths of perforated shearwall segments divided by the total width of the perforated shearwall including openings.

Figure 6: Shear Capacity adjustment factor - American Wood Council

Example values for Table-9 columns: *<u>For Row-3</u>*

- Max opening height = 5'-4" or 5.33'
- Length of building = 32'
- Window width (from CAD file provided) = 3'-8" or 3.66'
- Number of windows = 2
- Length of full height segment = (32' – 2x3.66') = 24.68'
- Percentage of full height segment = 24.68'/32' = 0.77 or 77%
- Shear adjustment factor from Fig.5 = 0.77 (use column *2h/3* in Fig.5)
- Adjusted capacity = 280 *plf* x 1.4 x 0.82 x 0.77 = 247.51 *plf*

- Unit shear in full height segments = 3526.77 *lbs* / 24.68 *ft* = 142.90 *plf*
- Check (adjusted capacity > Unit shear) = 247.51 plf > 142.90 plf = **OK**

Based on the calculation sample and the shear check provided in **Table 9**, the number of windows permitted for each shed size along with the window dimensions considering the minimum spacing requirement of 28 inches as explained in Section **4.1** were provided in **Table 10**. The maximum number of windows provided are only applicable for the height and width values provided in the table. Change in the window dimensions may alter the maximum number of windows allowed per wall.

Table 9: Window/Openings Requirements for Shear

Max opening height (ft)	Bldg. length (ft)	Window width (ft)	# Of windows	Length of full height segment (ft)	% Full - height	Shear adjustment factor, C_o	Adjusted capacity of openings (plf)	Unit shear in full height segments (plf)	Check
2.67	10	2.00	1	8.00	80%	1.00	321.44	312.72	OK
4.00	28	3.66	2	20.68	74%	0.87	279.65	145.03	OK
5.33	32	3.66	2	24.68	77%	0.77	247.51	142.90	OK
5.33	40	3.66	2	32.68	82%	0.83	266.80	125.58	OK

Table 10: Window Schedule per building size

Shed Size.	Max Window Height	Window Width	Max windows per wall
8’x10’	2’-8”	2’-0”	1
10’x28’	4’-0”	3’-8”	2
12’x32’	5’-4”	3’-8”	2
14’x40’	5’-4”	3-8”	3

5 Wall Stud Requirements

Using the loads from C&C the stud requirements were assessed in this section. The stud spacing, span lengths, and dimensions are obtained from the CAD file provided.

- Load, *L* = -73.70 psf (worst case from **Table 4**)
- Clear span = 8 ft
- Spacing = 16 in
- Size = 2x4
- Section module = 3.06 in^3 (for 2x4 stud)
- Inertia = 5.36 in^4 (for 2x4 stud)

5.1 Stress Checks

5.1.1 Applied load

$$W = (spacing)(L) = 1.33ft * 73.70psf = 98\,plf$$

5.1.2 Bending stress

$$Mmax = \frac{Wl^2}{8} = \frac{(98\,plf)(8^2)}{8} = 784\,ft - lb$$

$$F_b = \frac{M}{S} = \frac{(784\,ft - lb)(12\frac{in}{ft})}{3.06\,in^3} = 3074.5\,in - lb$$

5.1.3 Horizontal shear stress

$$Vmax = \frac{Wl}{2} = \frac{(98\,plf)(8ft)}{2} = 392lbs$$

$$f_v = \frac{3V}{2A} = \frac{(3)(392)\,lbs}{2(1.5\,in)(3.5in)} = 111\,psi$$

5.1.4 Bending stress

$$R1 = R2 = Vmax = 392\,lbs$$

$$fc\perp = \frac{R}{A_b} = \frac{392lbs}{(2in)(1.5in)} = 130.66\,psi$$

5.1.5 Modulus for deflection criteria

For deflection criteria, FBC 2020 specifies that the wind load from “Component & Cladding” (C&C) loading can be adjusted with a factor of ***0.6*** to determine the deflection. This particular factor is applicable when the members support glass. The possibility of windows being supported by the members may account for the glass and hence the C&C load is adjusted by *0.6 factor*.

$$\rho_{max} = \frac{5wl^4}{384EI} = \frac{5(98\,x0.6\,plf)(8ft)^4\left(1728\frac{in^3}{ft^3}\right)}{384\,E(5.36\,in^4)} = \frac{1{,}011{,}009}{E}$$

$$\rho_{allow} \leq \frac{l}{180}\ (for\ walls)$$

$$\rho_{max} \leq \rho_{allow}$$

$$\frac{1{,}011{,}009}{E} \leq \frac{(8ft)\left(12\frac{in}{ft}\right)}{180}$$

$$E_{min} = 1.89\ x10^{6}\ psi$$

5.1.6 Joist/Stud and Wall Sheathing Combination

However, the wall framing studs and the wall sheathing together resist the wind forces. Therefore, for the deflection calculations, the inertia can be adjusted to account for the wall sheathing. **Table 11** provides the values for a typical *SPF grade #2 2x4 stud* with 16" OC spacing and structural sheathing with nominal thickness of *0.375in.* The combined moment of inertia for the stud and sheathing is **8.88in^4.**

Table 11: Combined properties of Joist and Wall Sheathing

	Area (in^2)	d (in)	Area x d (in^3)	y (in)	Ay^2 (in^2)	Moment of Inertia (in^4)	Combined Moment of Inertia (in^4)
Sheathing	1.14	3.69	4.21	1.59	2.89	0.002	-
Joist	5.25	1.75	9.19	0.35	0.63	5.36	-
Totals	**6.39**		**13.40**		**3.52**	**5.36**	**3.52+5.36 = 8.88**

Neutral Axis (*NA*) of the combination (joist + sheathing) = $\frac{Total(Area\ x\ d)}{Total(Area)} = \frac{13.40}{6.39} = 2.10\ in$

'Y' values in **Table 11**:

- For sheathing : $y = d - NA = 3.69 - 2.10 = 1.59\ in$
- For Joist : $y = NA - d = 2.10 - 1.75 = 0.35\ in$

The deflection check with modified moment of inertia (joist + sheathing):

$$\rho_{max} = \frac{5wl^4}{384EI} = \frac{5(98x0.6\ plf)(8ft)^4\left(1728\frac{in^3}{ft^3}\right)}{384\ E(8.88\ in^4)} = \frac{610{,}249}{E}$$

$$\frac{610{,}249}{E} \leq \frac{(8ft)\left(12\frac{in}{ft}\right)}{180} = E_{min} = 1.14x10^{6}\ psi$$

5.2 Minimum unadjusted properties from National Design Specification (NDS) for wood construction.

5.2.1 Bending

$f_b \leq F_b'$

$F_b' = F_b * C_r * C_F * C_D$

$$F_{\ bmin}@175mph = \frac{f_b}{C_r C_F C_D} = \frac{3074.5psi}{(1.15)(1.1)(1.6)} = 1519\ psi$$

Similar to deflection, the bending f_b values are adjusted for the joist and plywood combination. The section modulus for joist and plywood combination is ***S = 4.44in³***. Therefore, the updated F_b

$$F_b = \frac{M}{S} = \frac{(784\ ft - lb)(12\frac{in}{ft})}{4.44\ in^3} = 2119\ in - lb$$

$$F_{\ bmin}@175mph\ = \frac{2119\ psi}{(1.15)(1.1)(1.6)} = 1046\ psi$$

$$F_{\ bmin}@160mph\ = \frac{1771\ psi}{(1.15)(1.1)(1.6)} = 875\ psi$$

5.2.2 Horizontal Shear

$f_v \leq F_v'$

$F_v' = F_v * C_H * C_D$

$$F_{vmin}\ = \frac{f_v}{C_H C_D} = \frac{111\ psi}{(2)(1)}\ =\ 55.5\ psi$$

5.2.3 Bearing

$f_{c\perp} \leq F_{c\perp}'$

$F_{c\perp} = F_{c\perp} * C_b$

$$F_{c\perp min}\ = \frac{f_{c\perp}}{1.0}\ = \frac{130.66\ psi}{1.0}\ = 130.66\ psi$$

5.2.4 Minimum unadjusted properties required by stud materials

- F_b = 1046 psi @ 175mph & 875 psi @ 160mph
- F_v = 55.5 psi
- $F_{C\perp}$ = 130.66 psi

5.2.5 Adjustment factors used in the above calculations per NDS

- C_F, Size Factor = 1.1
- C_D, Load Duration Factor = 1.6
- C_r, Representative Member Factor = 1.15
- C_H, Horizontal Shear Factor = 2.0
- C_b, Bearing Area Factor = 1.0

5.3 Studs Material Recommendation:

Deflection Criteria

- **Spruce-Pine-Fir (SPF)** studs with E = 1.5x10^6 psi for wind speeds up to **155mph.**
- **SPF studs** with **structural sheathing** with min **nominal thickness of *0.375in*** can be used for wind speeds up to **175mph.**
- **Southern Yellow Pine (SYP)** studs with E = 1.5x10^6 psi can also be used for speeds up to **175mph**.

Bending Criteria

- **SPF** studs with F_b = 875 psi for wind speeds up to ***160mph***
- **SYP** studs with F_b greater than *1100 psi* for wind speeds higher than ***160mph up to 175mph***

6 Roof Truss Design

Max wind load from ASCE 7-16 fig 28.5-1 = 61.7psf (uplift @ 180mph)

$$Lmax = 84" = 7ft\ for\ 2x4\ @24"$$

$$W = (spacing)(L) = 2ft * 61.7psf = 123.4\ plf$$

$$Mmax = \frac{Wl^2}{8} = \frac{(123.4\ plf)(7^2)}{8} = 755\ ft - lb$$

$$f_b = \frac{M}{S} = \frac{(755\ ft - lb)(12\frac{in}{ft})}{3.06\ in^3} = 2964\ in - lb$$

F_b(LRFD) = 3000 psi

f_b=2964 psi < 3000 psi - **OK**

The collar placed at 2/3rd of the vertical distance between top plate and the ridge will reduce the span length, L_{max} to 5ft.

$$Mmax = \frac{Wl^2}{8} = \frac{(123.4\ plf)(5^2)}{8} = 385\ ft - lb$$

$$f_b = \frac{M}{S} = \frac{(385\ ft - lb)(12\frac{in}{ft})}{3.06\ in^3} = 1512\ \ in - lb$$

F_b(LRFD) = 3000 psi

f_b=1512 psi < 3000 psi - **OK**

6.1 Rafter Material Recommendation:

- **2x4 SYP No.2** rafters with F_b > ***1500 psf*** can be used for wind speeds up to **180mph.**

7 Floor Joist Design

The max floor live loads provided in the CAD file shared with KiloNewton were used in this section. The stud spacing, span lengths, and dimensions are obtained from the CAD file provided.

- Load, *L* = 130 psf (live load 120psf and dead load 10psf)
- Clear span = 6 ft (max span allowed per CAD)
- Spacing = 16 in
- Size = 2x4
- Section module = 3.06 in^3 (for 2x4 stud)
- Inertia = 5.36 in^4 (for 2x4 stud)

7.1 Stress Checks

7.1.1 Applied load

$$W = (spacing)(L) = 1.33ft * 130\,psf = 172.9\,plf$$

7.1.2 Bending stress

$$Mmax = \frac{Wl^2}{8} = \frac{(172.9\,plf)(6^2)}{8} = 778\,ft - lb$$

$$F_b = \frac{M}{S} = \frac{(778\,ft - lb)\left(12\frac{in}{ft}\right)}{3.06\,in^3} = 3051\,in - lb$$

7.1.3 Horizontal shear stress

$$Vmax = \frac{Wl}{2} = \frac{(173plf)(6ft)}{2} = 519\,lbs$$

$$f_v = \frac{3V}{2A} = \frac{(3)(519)\,lbs}{2(1.5\,in)(3.5in)} = 148psi$$

7.1.4 Bending stress

$$R1 = R2 = Vmax = 519\,lbs$$

$$fc\perp = \frac{R}{A_b} = \frac{519lbs}{(2in)(1.5in)} = 173\,psi$$

7.1.5 Modulus for deflection criteria

For deflection criteria, using the combination of plywood (minimum nominal thickness = 0.75in) and joist (2x4) and the combination moment of inertia estimated as explained in Section 5.1.6. The floor live load used here is *110psf* and the corresponding *W* is *146psf.*

$$\rho_{max} = \frac{5wl^4}{384EI} = \frac{5(146\,plf)(6ft)^4\left(1728\frac{in^3}{ft^3}\right)}{384\,E(12.55\,in^4)} = \frac{339{,}231}{E}$$

$$\rho_{allow} \le \frac{l}{360}\ (for\ floors)$$

$$\frac{339{,}231}{E} \leq \frac{(6ft)\left(12\frac{in}{ft}\right)}{360} = E_{min} = 1.69x10^6\ psi$$

7.2 Minimum unadjusted properties from National Design Specification (NDS) for wood construction.

7.2.1 Bending

$f_b \leq F_b'$

$F_b' = F_b * C_r * C_F * C_D$

$$F_{\ bmin}@175mph = \frac{f_b}{C_r C_F C_D} = \frac{3051\ psi}{(1.15)(1.1)(1.6)} = 1508\ psi$$

As explained in Section 5.1.6, the bending f_b values are adjusted for the joist and plywood combination (with a minimum nominal thickness of 0.75in). The section modulus for joist and plywood combination is approximately ***S = 6.27in³***.

Therefore, the updated F_b

$$F_b = \frac{M}{S} = \frac{(778\ ft - lb)(12\frac{in}{ft})}{6.27\ in^3} = 1489\ in - lb$$

$$F_{\ bmin} = \frac{1489\ psi}{(1.15)(1.1)(1.6)} = 735\ psi$$

7.2.2 Horizontal Shear

$f_v \leq F_v'$

$F_v' = F_v * C_H * C_D$

$$F_{vmin} = \frac{f_v}{C_H C_D} = \frac{148\ psi}{(2)(1)} = 74\ psi$$

7.2.3 Bearing

$f_{c\perp} \leq F_{c\perp}'$

$F_{c\perp} = F_{c\perp} * C_b$

$$F_{c\perp min} = \frac{f_{c\perp}}{1.0} = \frac{173\ psi}{1.0} = 173\ psi$$

7.2.4 Minimum unadjusted properties required by stud materials

- F_b = 1039 psi @ 175mph & 868 psi @ 160mph
- F_v = 74 psi
- $F_{C\perp}$ = 173 psi

Note: the floor sheathing should have a min nominal thickness of *0.375in*. The CAD plans specify 3/4 or *0.75in* plywood which is more than adequate.

7.2.5 Adjustment factors used in the above calculations per NDS

- C_F, Size Factor = 1.1
- C_D, Load Duration Factor = 1.6
- $C_{r,}$ Representative Member Factor = 1.15
- C_H, Horizontal Shear Factor = 2.0
- C_b, Bearing Area Factor = 1.0

7.3 Studs Material Recommendation:

Deflection Criteria

- **Spruce-Pine-Fir (SPF)** studs with **E = 1.5x10^6 psi** and structural sheathing with a minimum **nominal thickness of 0.75in** can be used for floor live loads up to **100psf.**
- **Southern Yellow Pine (SYP)** studs with **E = 1.7x10^6 psi** and structural sheathing with a minimum **nominal thickness of 0.75in** can be used for floor live loads up to **110psf**.
- **Southern Yellow Pine (SYP)** studs with **E = 1.8x10^6 psi** or higher and structural sheathing with a minimum **nominal thickness of 0.75in** or greater are needed for floor live loads up to **120psf**.

Bending Criteria

- **SPF** or **SYP** studs with $F_b > 800$ *psi* along with structural sheathing of nominal thickness of ***0.75in*** or greater can be used for floor live loads up to ***120psf.***

8 Fastener Checks

8.1 Shear wall tie-downs

The shear wall calculations are performed assuming that the hold-down (aka tie-down) anchors provide necessary support. The support needed per shear wall will be equal to the load per wall estimated. The max spacing for tie-down anchors per CAD plan provided is 8ft and therefore the controlling load for shear wall hold-down requirement is 2501lbs (calculated in Section 4.1 **Table 8**).

- Hold-down capacity required at **175mph** = 2501lbs
- Hold-down capacity required at **170mph** = 2359lbs

The following tie-down anchors have different hold-down capacities:

- 1/2in x 30in eye anchor with 4in helix: **2400 lbs**
- 5/8in x 40in eye anchor with 6 in helix: **4,000 lbs**

Therefore, 1/2in x 30in eye anchor with 4in helix can be used as hold-down anchors with no more than 8ft spacing up to wind speeds **170mph**. For speeds greater than 170mph, 5/8in x 40in anchor with 6in helix need to be used.

8.2 Wall to Floor connection

- <u>Type of connection</u>

  Simpson H2.5A at each rafter w (4) 8D nails (From CAD designs shared with KiloNewton)

- <u>Load to be supported by the connection</u>

  Max wind load from **Table *4*** = 73.7 psf (@ 175mph)

$$W = (spacing)(L) = 1.33ft * 73.70psf = 98\,plf$$

$$W\ @\ each\ end\ of\ the\ stud = 98\,plf\ x\ 4ft = 392lbs$$

- <u>Capacity of the connection</u>

  Simpson H2.5A at each stud w/ 8d x1.5" nails (uplift capacity from Simpsons product manual)
  - **SPY**: 625 lbs > 392 lbs – **OK**
  - **SPF**: 540 lbs > 392 lbs – **OK**

8.3 Wall to Roof connection

- <u>Type of connection</u>

  Simpson H1 at each rafter w (4) 8d nails (From CAD designs shared with KiloNewton)

- <u>Load to be supported by the connection</u>

  Max wind load from ASCE 7-16 fig 28.5-1 = 61.7psf (uplift @ 180mph)

$$Lmax = 84" = 7ft\ for\ 2x4\ @24"\ O.C.$$

$$W = (spacing)(L) = 2ft * 61.7psf = 123.4\,plf$$

$$W\ @\ each\ end\ of\ the\ rafter = 123.4\,plf\ x\ 3.5ft = 432\ lbs$$

- <u>Capacity of the connection</u>

  Simpson H1 at each rafter w/ 8D nails (uplift capacity from Simpsons product manual):
  - **SPY**: 600 lbs > 432 lbs – **OK**
  - **SPF**: 500 lbs > 432 lbs – **OK**

8.4 Roof to Roof connection (Ridge)

- <u>Type of connection</u>

  2x4 Truss Plate (From CAD designs shared with KiloNewton)

- <u>Load to be supported by the connection</u>

  The load acting on the truss plate comes from either of the roof rafter connected by the truss plate. Each rafter carries the *W* estimated in the calculation above (i.e., *W* @ end of rafter in Section 8.3). Therefore, the total load acting on the truss plate is *2x432lbs = 864lbs.* However, the collar ties reduce the span length of the rafter and thereby, the load acting on the ends of the rafters.

The adjusted load at the end of the rafter will be:

$$W \ @\ each\ end\ of\ the\ rafter = 123.4\ plf\ x\ 2.5ft\ = 309\ lbs$$

$$Total\ W\ @\ Truss\ plate = 309\ lb\ x\ 2\ = 618\ lbs$$

- Capacity of the connection

  Simpson 2x4 Truss plate (Truss Bearing Enhancer, TBE) (uplift capacity from Simpsons product manual):

  - **SPY rafters**: 730 lbs > 618 lbs – **OK (**Roof rafters are always **SYP)**