Job#:	**2015-009**
Engr:	NPW
Date:	4/4/2015

Post Frame Building

Building Specifications:

Building Length	30	ft	Roof Overhang Eave	24	in
Building Width	20	ft	Roof Overhang Gable	24	in
Post/Wall Height	14	ft	Total Roof Height	4.00	ft
Post Spacing (sidewall)	10	ft	Total Roof Length	34	ft
Roof Pitch	4	:12			
Roof Angle	18.43	deg	Post Spacing (endwall)	20	ft
Roof Height	3.33	ft			

Post Properties:

Post Width	5.50	in	Concrete Back fill	YES
Post Depth	5.50	in		
Post Size, Species and Grade	6X6 HF No. 1			

Post Design Values (constrained post design assumed)

F_b	975	psi			
F_c	850	psi	Post Hole Dia./Footing (sidewall)	18	in
$F_{c\perp}$	405	psi	Embedment Depth (sidewall)	4	ft
E	1,300,000	psi	Wall A		
E_{min}	470,000	psi	Post Hole Dia./Footing (endwall)	18	in
C_{F_b}	1.00		Embedment Depth (endwall)	4	ft
C_{F_c}	1.00		Wall B		
A	30.25	in^2	Post Hole Dia./Footing (endwall)	18	in
S_x	27.73	in^3	Embedment Depth (endwall)	4	ft
I_x	76.26	in^4			

Temp/Moisture/Incised (Typically these values are set to default of 1.0)

Ct_c	1.00
CM_c	1.00
Ci_c	0.80
Ct_b	1.00
CM_b	1.00
Ci_b	0.80
Ct_e	1.00
CM_e	1.00
Ci_e	0.95

ASD Wind Pressures (Transverse) [Values from MWFRS Wind Calculator]

Basic Wind Speed (Ultimate)	135.00	mph	Exposure: B
q_{ww}	9.30	psf	Windward Wall
q_{lw}	-5.88	psf	Leeward Wall
q_{wr}	-1.49	psf	Windward Roof (Positive)
q_{lr}	-6.90	psf	Leeward Roof

Total Wind Pressures (Transverse)

Wind Pressure Walls (side walls)	15.18	psf	
Wind Pressure Roof	5.41	psf	
Max. Dist. Load on Sidewall Posts (w)	**7.75**	lbs/in	(windward wall pressure governs)

Max. Diaphragm Unit Shear (Rigid Diaphragm) (Transverse)

Fixity Factor (f)	0.375		(3/8 for embedded posts)
v (fixed method)	78	plf	
V (fixed method)	1,564	lbs	
W_D (fixed method)	104	plf	
v (pin/roller method @ .7d)	**82**	plf	0.396 (.7d pin/roller factor)
V (pin/roller method @ .7d)	**1,630**	lbs	
W_D (pin/roller method @ .7d)	**109**	plf	

Diaphragm Deflection (Transverse) [based on a uniform distrubtion of eave loading and shearwall reactions]

Roof Diaphragm Length	30	ft	Mmax	12,227	ft-lbs
Unit Shear	82	plf	T=C=M/b=	611	lbs

Diaphragm Deflection (based on asd-level wind forces) (assume 8d common nails)

vu =	81.5	plf	nail spacing	6	in
Diaphragm Chord =	5-1/8"x10-1/2" GLB		Blocked	NO	
Chord Height =	10.500	in	en base	616	(Table C4.2.2D)
Chord Width =	5.125	in	en exp	3.018	(Table C4.2.2D)
E =	1,600,000	psi	Structural I	1.2	(footnote 1)
A =	53.81	in^2	V_n	40.8	lbs/nail (max. 220 lbs/nail)
G_vt_v =	83,500	lbs/in (Table C4.2.2A)	nail dia.	0.162	in (chord splices)
Δc =	0.038	in (per NDS C4.2.2-3)	x	10	ft
e_n =	0.0003	in (Table C4.2.2D)	γ	11,737	lbs/in/nail
# of chord splices =	2		# of nails req.	2.7	nails (chord splices)

Δb	Δv	Δn	Δa	**Δd**
0.0008	0.0122	0.0031	0.0384	**0.0546**

Endwall Shearwall Deflections:

Wall A Openings Width	12	ft
Wall B Openings Width	0	ft

Distribution of Shear to Endwalls: RIGID

SWL A Deflection (based on asd-level wind forces) (assume 8d common nails)

SWL A Length	8	ft		
Unit Shear	102	plf	V_w =	**813** lbs
SWL Chord Uplift	1,422	lbs		

vu =	101.6	plf	nail spacing	4	in
E =	1,300,000	psi	Sht. Both sides	NO	
A =	30.25	in^2	en base	616	(Table C4.2.2D)
G_vt_v =	35,000	lbs/in (Table C4.2.2A)	en exp	3.018	(Table C4.2.2D)
Δa =	0	in (Simpson Holdown or N/A)	Structural I	1.2	(footnote 1)
e_n =	0.0002	in (Table C4.2.2D)	V_n	33.9	lbs/nail (max. 220 lbs/nail)

Δb	Δv	Δn	Δa	**Δsw**
0.0071	0.0406	0.0020	0.0000	**0.0497**

SWL B Deflection (based on asd-level wind forces) (assume 8d common nails)

SWL B Length	20	ft		
Unit Shear	122	plf	V_w =	**2,448** lbs
SWL Chord Uplift	1,714	lbs		

vu =	122.4	plf	nail spacing	6	in
E =	1,300,000	psi	Sht. Both sides	NO	
A =	30.25	in^2	en base	616	(Table C4.2.2D)
G_vt_v =	35,000	lbs/in (Table C4.2.2A)	en exp	3.018	(Table C4.2.2D)
Δa =	0	in (Simpson Holdown or N/A)	Structural I	1.2	(footnote 1)
e_n =	0.0011	in (Table C4.2.2D)	V_n	61.2	lbs/nail (max. 220 lbs/nail)

Δb	Δv	Δn	Δa	**Δsw**
0.0034	0.0490	0.0119	0.0000	**0.0642**

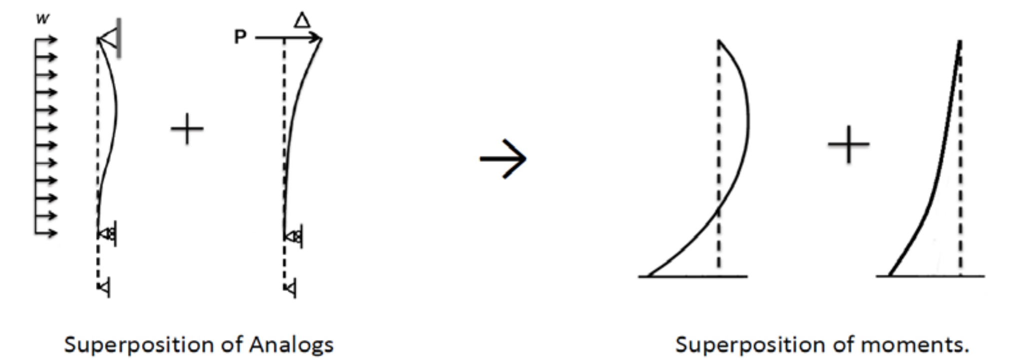
Δsw_avg = 0.057 in

Δeave = Δd + Δsw_avg = **0.112** in

Δd = 0.055 < 2(Δsw_avg) = 0.114 (Rigid Diaphragm) [ASCE 12.3.1.3]

Post Moments (Transverse)

Pin/Roller Ground Line Condition



Assume maximum post moment occurs in central frame of windward post with wind from transverse direction:

Post/Wall Height	14 ft	Max. Distributed Load on Post (w)	7.75 lbs/in
E	1,300,000 psi	Embedment Depth	4 ft
Ix	76.26 in^4	Δeave = Δd + Δsw_avg =	0.112 in

1st term: 27,339 — 2nd term: 1,175

Ground Line Moment (fixed method) $M_{max}^{(-)}$ = 28,515 in-lbs

$$M_{max} = \frac{wh^2}{8} + \frac{3EI\Delta_{eave}}{h^2}$$

1st term: 22,783 — 2nd term: 979

Ground Line Moment (pin/roller method) $M_{max}^{(-)}$ = **23,762** in-lbs

$$M_{max} = \frac{3EI\left[\dfrac{\frac{wh^2}{8}}{\frac{3EI}{h} + \frac{3EI}{0.7d}}\right]}{0.7d} + \frac{3EI\Delta_{eave}}{h[0.7d + h]}$$

1st term: 15,378 — 2nd term: 441

Max. Positive Moment (fixed method) $M_{max}^{(+)}$ = 14,938 in-lbs

$$M_{max} = \frac{9wh^2}{128} - \frac{9EI\Delta_{eave}}{8h^2}$$

Design for Lateral Loading:

Post Embedment of Central Frame Post (windward post)

Assume constrained post w/o collar

Allowable Lateral Soil Bearing Pressure (S)	150 psf/ft
Lateral Adj. Factors (per IBC Section 1806.3)	1.00
Adj. Allowable Lateral Soil Bearing Pressure (S')	150.0 psf/ft
Bending Moment at Ground Line (M_a)	1,980 ft-lbs
Concrete Back fill	YES
Effective Post Width	1.50 ft

$$d_{embedd} = \sqrt[3]{\frac{4.25Ma}{S'b}}$$

d_{embedd} = 3.34 ft < 4 ft ⟶ **OK**

Vertical Loads

Snow loads are calculated with the snow load calcutor per ASCE 7-10 Chapter 7:

Balanced Snow Load (p_s)	19.3	psf
Top Chord Dead Load (TCDL)	10.0	psf
Bottom Chord Dead Load (BCDL)	0.0	psf
Exterior Wall Dead Load (WDL)	0.0	psf
Windward Roof Load (Neg. Int. Pressure)	1.00	psf

$Area_{trib}$	Roof [ft^2]	Wall [ft^2]
Snow	120	0
TCDL	120	0
BCDL	100	0
WDL	0	140
Wind	120	0

(overhang uplift conservatively neglected)

Slope Factor (SF) 1.05
adj. TCDL 10.54 psf

Central Frame Post

Dead Load	1,265	lbs
Snow Load	2,316	lbs
Wind Load	121	lbs

ASD Load Cases from ASCE 7-10:

1.) D =	1,265	lbs
3.) D + (Lr or S) =	3,581	lbs
5.) D + (0.6W) =	1,385	lbs
6a.) D + .75L(0.6W) + .75(Lr or S) =	3,092	lbs

Corner Post

Dead Load	885	lbs
Snow Load	1,621	lbs
Wind Load	84	lbs

$Area_{trib}$	Roof [ft^2]	Wall [ft^2]
Snow	84	0
TCDL	84	0
BCDL	50	0
WDL	0	210
Wind	84	0

(overhang uplift conservatively neglected)

ASD Load Cases from ASCE 7-10:

1.) D =	885	lbs
3.) D + (Lr or S) =	2,507	lbs
5.) D + (0.6W) =	970	lbs
6a.) D + .75L(0.6W) + .75(Lr or S) =	2,165	lbs

Design for Downward Loading:

Post Footing of Central Frame Post (windward post)

Allowable Vertical Soil Bearing Pressure (S_v)	1500	psf
Vertical Adj. Factors (ASAE EP486.1)	1.60	(see notes below)
Adj. Allowable Lateral Soil Bearing Pressure (S_v')	2400	
Vertical Foundation Load (P)	3,581	lbs
Actual Area of Footing (A_{ftg})	1.77	ft^2

A_{req} = 1.49 ft2 < 1.77 ft^2 ⟶ **OK**

Actual Area of Footing (A_{ftg}) - Wall A 1.77 ft^2
Actual Area of Footing (A_{ftg}) - Wall B 1.77 ft^2

Post Footing of Corner Post (windward post)

Allowable Vertical Soil Bearing Pressure (S_v)	1500	psf
Vertical Adj. Factors (ASAE EP486.1)	1.60	(see notes below)
Adj. Allowable Lateral Soil Bearing Pressure (S_v')	2400	
Vertical Foundation Load (P)	2,507	lbs
Actual Area of Footing (A_{ftg}) [min. of A and B]	1.77	ft^2

A_{req} = 1.04 ft2 < 1.77 ft^2 ⟶ **OK**

Notes:
Allowable foundation pressures are for footings at least 300 mm (1 ft) wide and 300 mm (1 ft) deep into natural grade. Pressure may be increased 20% for each additional 300 mm (1 ft) of width and/or depth to a maximum of three times the tabulated value.

ASD Wind Pressures (Longitudinal)	[Values from MWFRS Wind Calculator]		
Basic Wind Speed (Ultimate)	135.00	mph	Exposure: B
q_{ww}	9.30	psf	Windward Wall
q_{lw}	-4.71	psf	Leeward Wall
q_{ww}	10.10	psf	Windward Wall
q_{lw}	-4.71	psf	Leeward Gable Wall

Total Wind Pressures (Longitudinal)			
Wind Pressure Walls (end walls)	14.01	psf	
Wind Pressure Walls (gable walls)	14.80	psf	
Max. Dist. Load on Endwall Posts (w)	**7.75**	lbs/in	(windward wall pressure governs)

Max. Diaphragm Unit Shear (Rigid Diaphragm)		(Longitudinal)	
Fixity Factor (f)	0.375		(3/8 for embedded posts)
v (fixed method)	**33**	plf	
V (fixed method)	**982**	lbs	
W_D (fixed method)	**98**	plf	

Design for Upward Loading: Load Combo: .6D + .6W

			$Area_{trib}$	Roof [ft^2]	Wall [ft^2]
Top Chord Dead Load (TCDL)	6.0	psf	TCDL	120	0
Bottom Chord Dead Load (BCDL)	0.0	psf	BCDL	100	0
Exterior Wall Dead Load (WDL)	0.0	psf	WDL	0	140
Max. Wind Uplift (Pos. Int. Pressure)	-10.31	psf	Wind	120	0
Windward Overhang Uplift	-9.30	psf	Wind	20	0

Central Frame Post

Dead Load	720	lbs
Wind Load	-1,423	lbs
ASD Load Cases from ASCE 7-10:		
7.) 0.6D + 0.6W =	**-703**	lbs
Concrete Back fill	YES	
Volume of Back fill	6.23	ft^3
Density of Back fill	150	pcf
Weight of Back fill + Soil Friction	**1,641**	lbs → **OK**

Central Frame Post
Uplift Variables as Applicable

ρ_{conc} =	150	pcf
k_{cs} =	0.25	
ρ_{soil} =	105	pcf
ϕ_{soil} =	30	deg
collar depth =	6	in

Design for Uplift at End Shearwalls: Load Combo: .6D + .6W

			$Area_{trib}$	Roof [ft^2]	Wall [ft^2]
Top Chord Dead Load (TCDL)	6.0	psf	TCDL	84	0
Bottom Chord Dead Load (BCDL)	0.0	psf	BCDL	50	0
Exterior Wall Dead Load (WDL)	0.0	psf	WDL	0	210
Max. Wind Uplift (Pos. Int. Pressure)	-3.98	psf	Wind	84	0
Windward Overhang Uplift	-9.30	psf	Wind	10	0

Corner Posts

Dead Load		504	lbs
Wind Load		-427	lbs
SWL A Chord Uplift		-1422	lbs
SWL B Chord Uplift		-1714	lbs
ASD Load Cases from ASCE 7-10:			
SWL A 7.) 0.6D + 0.6W =		**-1,345**	lbs
SWL B 7.) 0.6D + 0.6W =		**-1,637**	lbs
Concrete Back fill		YES	
Volume of Back fill	(SWL A)	6.23	ft^3
Volume of Back fill	(SWL B)	6.23	ft^3
Density of Back fill		150	pcf
Weight of Back fill + Soil Friction	(SWL A)	**1,641**	lbs → **OK**
Weight of Back fill + Soil Friction	(SWL B)	**1,641**	lbs → **OK**

Corner Post
Uplift Variables as Applicable

ρ_{conc} =	150	pcf
k_{cs} =	0.25	
ρ_{soil} =	105	pcf
ϕ_{soil} =	30	deg
collar depth A =	6	in
collar depth B =	6	in

Rigid Diaphragm Analysis (Transverse)

[Based on ASD Wind Pressures]

Diaphragm Loading (W_D)	109	plf
Total Lateral Load (F_{px})	3,261	lbs
(measured from wall line A)		
Center of Load (d_w)	15.0	ft
Center of Rigidity (d_r)	25.9	ft
Eccentricity (e)	10.9	ft
Design Eccentricity (e')	11.4	ft

Apparent Shear Stiffness Ga			Notes
SWL A	4	kips/in	RISA Frame Analysis
SWL B	10	kips/in	SDPWS Table 4.3A
SWL 1	10	kips/in	SDPWS Table 4.3A
SWL 2	10	kips/in	SDPWS Table 4.3A

[ASCE 7-10 Ch. 12]

Wall Line	Stiffness k (kips/in)	Distance r (ft)	k x r	k x r^2	kr/Σkr^2	Fpxe'(kr) / Σkr^2	Rd (lbs)	V (lbs)
A	2.3	25.9	59	1,529	0.010	363	450	813
B	14.3	4.1	59	245	0.010	363	2,811	2,448
1	21.4	10	214	2,143	0.035	1315	0	1,315
2	21.4	10	214	2,143	0.035	1315	0	1,315
			Total	6,059				

Shear Wall Force Distribution (Transverse)

SWL	Flexible	Rigid	Semi-Rigid	[lbs]
A	1,630	813	1,221	
B	1,630	2,448	2,039	

Shear Wall Force Distribution (Longitudinal)

SWL	Flexible Long. Loading	Rigid Transverse Loading	[lbs]
1	982	1,315	
2	982	1,315	
		(governs)	

SWL 1/SWL 2

SWL Length	30	ft
Unit Shear	44	plf
SWL Chord Uplift	614	lbs

Notes:

Assume that there are no major openings in sidewalls and that any minor openings are symetrical and do not affect the center of rigidity in the direction perpendicular to the sidewalls.

POST DESIGN CALCULATIONS

Post Width (dy)	5.50	in
Post Depth (dx)	5.50	in
Post Length (L)	14.00	ft
Post Spacing	10.00	ft
Post Species and Grade	6X6 HF No. 1	
Top/Sill Plt. Species	HF	

Design Values

Fb	975	psi
Fc	850	psi
Fc⊥	405	psi
E	1,300,000	psi
Emin	470,000	psi
CF_b	1.00	
CF_c	1.00	
A	30.25	in^2
Sx	27.73	in^3
Ix	76.26	in^4
Ct_c	1.00	
CM_c	1.00	
Ci_c	0.80	

Load Case 1: Gravity Loads Only

ly (unbraced length)	2.0	ft
lex (effective unbraced length)	11.20	ft
CD	1.15	(Snow Load)
(le/d)y	4.36	
(le/d)x	24.44	(governs)
E'min	446,500	psi
FcE	614.64	psi
Fc*	782.00	psi
c	0.80	sawn lumber
FcE/Fc*	0.786	
1 + FcE/Fc*/2c	1.116	
Cp	0.603	
Fc'	471.47	psi
fc	118.38	psi
CSI (axial)	**0.25**	**OK**

Bearing on Wall Plates or Beams

lb	5.50	in
Cb	1.00	(conservative)
Fc⊥'	405.00	psi
fc⊥	118.38	psi
CSI (bearing)	**0.29**	**OK**

Deflection

E'	1,235,000	psi
ΔWIND (MWFRS)*	0.35	in
L/d*	**474**	**OK**

*IBC 2015 Sec. 1604.3

Vertical Loads

3.) D + (Lr or S) =	3,581	lbs
6a.) D + .75L(0.6W) + .75(Lr or S) =	3,092	lbs
Buckling Length Coeff. (Ke)	0.8	
(NDS Table G1 of Appendix G)		

Lateral Loads (Wind MWFRS)

Wind Load (windward wall)	15.50	psf
MWFRS Wind Load ASD	9.30	psf
Wind Atrib	140.00	ft^2
W	1301.88	lbs
w	92.99	plf

Load Case 2: Gravity Loads and Lateral Loads

CD	1.60	(Wind/Seismic)
Mmax	1,980	ft-lbs
	23,762	in-lbs

Calculation of Beam Stability Factor:

lu	14.00	ft
lu/d	30.55	
le	25.76	ft
Rb	7.50	
Rb < 50	**OK**	
Eminy'	446,500	psi
FbE	9,533	psi
Fbx*	1,248	psi
FbE/Fb*	7.639	
1 + FbE/Fb*/1.9	4.547	
CL	0.993	
Cr	1.00	@ 10' O/C
Fbx'	1238.75	psi
fbx	856.94	psi
CSI (bending MWFRS)	**0.69**	**OK**

Combined Stress

(re-evaluate compression values with CD = 1.6)

FcEx	614.64	psi
FcE	614.64	psi
Fc*	1088.00	psi
c	0.80	sawn lumber
FcE/Fc*	0.565	
1 + FcE/Fc*/2c	0.978	
Cp	0.478	
Fc'	519.63	psi
fc	102.23	psi

$$\left(\frac{f_c}{F_c^{'}}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}^{'}}\right) = \mathbf{0.87}\ \textbf{OK}$$

Location: Central Sidewall Posts
Specification: **Use 6X6 HF No. 1 Grade @ 10' o/c**

Rev. 1.0.0 - 04/01/2015

Roof Diaphragm and Sheathing Calculations

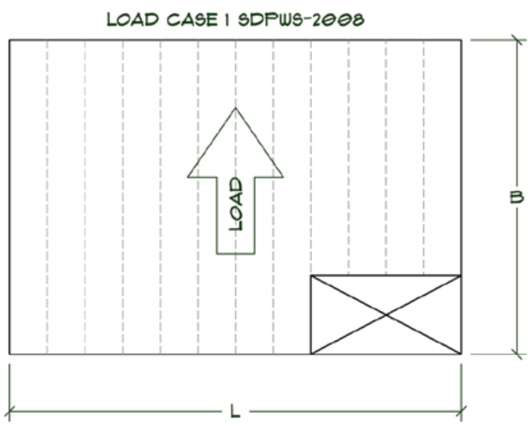
By inspection the highest stressed diaphragm is the main roof diaphragm. The transverse and longitudinal loads are obtained from previous calculations. We consider both cases and conservatively design for the worst load case.

1.) Transverse Load Case:

W_D =	109	plf	L =	30	ft
Mmax =	12,227	ft-lbs	b =	20	ft
T=C=M/b=	**611**	**lbs**			

V =	1630	lbs
v = V/b =	**82**	**plf**

Load Case 1
SDPWS-2008



L/B Ratio = 1.50 → **OK**

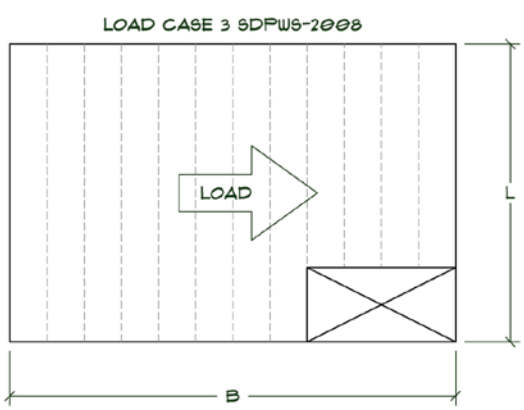
*Max. AR for WSP, unblocked = 3:1

2.) Longitudinal Load Case:

W_D =	98	plf	L =	20	ft
Mmax =	4,910	ft-lbs	b =	30	ft
T=C=M/b=	**164**	**lbs**			

V =	982	lbs
v = V/b =	**33**	**plf**

Load Case 3
SDPWS-2008



L/B Ratio = 0.67 → **OK**

*Max. AR for WSP, unblocked = 3:1

Roof Sheathing Specifications (Initially assume APA rated sheathing with nails @ 6" o/c edges, 12" o/c field.)

Sheathing Thickness:	7/16 in.	APA Rating:	Grade 24/16	
Nails:	8d	Sheathing Type:	OSB	
Rafter/Truss Spacing:	24 in. o/c	Roof Framing Species:	HF	
(Unblocked Diaphragm)		SGAF:	0.93	

Load Case 1: (transverse)

$v =$ 82 plf < $v_w =$ 300 plf ⟶ **OK**

Load Case 3: (longitudinal)

$v =$ 33 plf < $v_w =$ 221 plf ⟶ **OK**

Note: Nominal unit shear capacities for unblocked diaphragms from Table 4.2C, SDPWS-2008.

> Sheath roof with 7/16 APA rated OSB (Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field. Blocking not required at panel edges.

Chord Splices

From previous, transverse load case governs with largest chord force:

T = C = **611** lbs

Assume a min. 48" chord splice at top plate connected with two or three rows of 16d nails (.162" x 3.5").

From NDS 2012 Table 11N: $C_D =$ 1.6 (wind/seismic)

Z = 141 lbs

Z' = $Z(C_D) =$ 225.6 lbs

N = T/Z' = 2.7 nails

This number is too low, revert to prescriptive method: [Table 3.21 WFCM 2012]

> Use (8) - 16d nails on each side of splice joint in wall top chords. Position splice joint over studs.

> Where top chord is discontinous, apply an MSTC28 strap to complete the tensile load path. (ie. Where a beam ties into a top plate)

Roof Panel Sheathing Loads

Highest loading on roof sheathing panels is at roof overhangs in Zone 3 (C&C Wind Loads) with negative pressure/uplift.

Terrain Exp. Category: B
Basic Wind Speed (ultimate: 135.00 MPH

P_{3OH} = 91.0 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{3OH_ASD} : 54.6 psf

Roof Sheathing Nailing

	Edges (in.)	Field (in.)
Interior (Zone 1)	6	12
Perimeter (Zone 2)	6	6
Gable Endwall & Overhangs	6	6

*Based on WFCM 2012 Table 3.10, Rafter/Truss spacing @ 24" o/c.

Also consider highest gravity loads:

D + S (ice dam at overhangs)

P_S = 10.5 psf + 38.5 psf = 49.0 psf

Wind Load Governs: C_D = 1.6

From SDPWS-2008 Table 3.2.2 (Load Capacities for Roof Sheathing Resisting Out-of-Plane Loads):

Pmax = 54.6 psf < P_{allow} = 84.4 psf ⟶ **OK**

Also from APA publication Q225G Table 2a (OSB Sheathing):

Sheathing Perpendicular to Rafters/Trusses

L/240	⟶	51 psf	>	38.2 psf [2]	⟶ **OK**
L/180	⟶	68 psf	>	38.2 psf [2]	⟶ **OK**
Bending	⟶	128 psf	>	54.6 psf	⟶ **OK**
Shear	⟶	213 psf	>	54.6 psf	⟶ **OK**

*Note: L/240 is (live load) deflection, L/180 is (total load) deflection.

Install "h" clips at panel edges @ 24" o/c for all roof sheathing.

Nail all sheathing at gable and eave roof overhangs w/ 8d nails @ 6" o/c edges, 6" o/c field.
Nail all sheathing at perimeter and peak of roof w/ 8d nails @ 6" o/c edges, 6" o/c field.

General Notes:

1.) For roof sheathing within 4 feet of the perimeter edge of the roof, including 4 feet on each side of the roof peak, the 4 foot perimeter edge zone attachment requirements shall be used.

2.) The wind loading is permitted to be taken as 0.42 times the C&C loads for the purpose of determining deflection limits per footnote f. of Table 1604.3 IBC 2015.

Wall Sheathing Specifications (Initially assume APA rated sheathing with nails @ 6" o/c edges, 12" o/c field.)

Sheathing Thickness:	15/32	in.	APA Rating:	Grade 32/16
Nails:	8d		Sheathing Type:	PLY
Stud Spacing:	16	in. o/c		
			Terrain Exp. Category	B
			Basic Wind Speed (ultimate	135.00 MPH

Wall Panel Sheathing Loads

Highest loading of wall sheathing panels is at building corners in Zone 5 (C&C Wind Loads) with negative pressure/suction.

P_5 = 38.9 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{5_ASD} = 23.4 psf

Wall Sheathing Nailing

	Edges (in.)	Field (in.)
Interior (Zone 4)	6	12
Edge (Zone 5)	6	12

*Based on WFCM 2012 Table 3.11, Stud spacing @ 16" o/c.

From SDPWS-2008 Table 3.2.1 (Load Capacities for Wall Sheathing Resisting Out-of-Plane Loads):

Sheathing Parallel to Studs

P_{5_ASD} = 23.4 psf < P_{allow} = 56.3 psf → **OK**

Sheathing Perpendicular to Studs

P_{5_ASD} = 23.4 psf < P_{allow} = 221.9 psf → **OK**

Also from APA publication Q225G Table 1a (Plywood Sheathing):

Sheathing Parallel to Studs

L/360	→	13 psf	>	16.4 psf [2]	→	**NG**
Bending	→	69 psf	>	23.4 psf	→	**OK**
Shear	→	762 psf	>	23.4 psf	→	**OK**

Sheathing Perpendicular to Studs

L/360	→	205 psf	>	16.4 psf [2]	→	**OK**
Bending	→	277 psf	>	23.4 psf	→	**OK**
Shear	→	442 psf	>	23.4 psf	→	**OK**

Sheath walls with 15/32 APA rated PLY (Grade 32/16) w/ 8d nails @ 6" o/c edges, 12" o/c field.

Nail all sheathing within 4 feet of wall corners w/ 8d nails @ 6" o/c edges, 12" o/c field.

General Notes:

1.) For wall sheathing within 4 feet of the corners, the 4 foot edge zone attachment requirements shall be used.

2.) The wind loading is permitted to be taken as 0.42 times the C&C loads for the purpose of determining deflection limits per footnote f. of Table 1604.3 IBC 2015.

Job#: **2015-009**

WIND (MWFRS)

Wind Analysis Method	Analytic Directional Procedure			ASCE 7-10 Fig. 27.4-1
Basic Wind Speed (ultimate)		135.00	MPH	
Topography Factor	K_{zt} =	1.00		ASCE 7-10 Fig. 26.8-1
Directionality Factor	K_d =	0.85		ASCE 7-10 Fig. 26.6-1
Gust Effect Factor	G =	0.85		ASCE 7-10 Sec. 26.9.1
Internal Pressure Coefficients	(GC_{pi}) =	0.18	-0.18	ASCE 7-10 Table 26.11-1
Roof Pitch		4.00	:12	18.43 DEG
Roof Eve Height		14.000	FT	
Peak Roof Height		17.333	FT	α = 7
Mean Roof Height		15.667	FT	z_g = 1200
Terrain Exp. Category		B		

Velocity Pressures

$q_z = .00256 K_z K_{zt} K_d V^2$

Height (ft)			K_z	q_z
h_e =	14.00	FT	0.575	22.79
h =	15.67	FT	0.582	**23.08**
z =	15	FT	0.575	22.79
z =	20	FT	0.624	24.74
z =	25	FT	0.665	26.37
z =	30	FT	0.701	27.78

L = Parallel to wind dir.
B = Perp. to wind dir.

Design Pressures

$p = qGC_p - q_h(GC_{pi})$

Transverse Direction:

Note: Pressures are limit state design pressures for strength design. Multiple by 0.6 for ASD.

L = 20 L/B = 0.67
B = 30 h/L = 0.78

Design Pressure (psf)

	z (ft)	q_z (psf)	Cp	qGC_p	(+GCpi)	(-GCpi)
Windward Wall	15	22.79	0.80	15.50	**11.34**	**19.65**
	20	24.74	0.80	16.83	**12.67**	**20.98**
	25	26.37	0.80	17.93	**13.78**	**22.09**
	30	27.78	0.80	18.89	**14.74**	**23.05**
Leeward Wall	15.67	23.08	-0.50	-9.81	**-13.96**	**-5.65**
Side Wall	15.67	23.08	-0.70	-13.73	**-17.88**	**-9.58**
Windward Roof (Positive)	15.67	23.08	-0.13	-2.48	**-6.63**	**1.67**
Windward Roof (Negative)	15.67	23.08	-0.66	-13.02	**-17.18**	**-8.87**
Leeward Roof	15.67	23.08	-0.59	-11.50	**-15.66**	**-7.35**
Ridge Parallel Roof	(0 to h/2)	23.08	-1.13	-22.10	**-26.25**	**-17.95**
	(h/2 to h)	23.08	-0.79	-15.43	**-19.58**	**-11.28**
	(h to 2h)	23.08	-0.61	-12.03	**-16.18**	**-7.88**
	(>h2)	23.08	-0.53	-10.33	**-14.48**	**-6.18**

Longitudinal Direction:

Note: Pressures are limit state design pressures for strength design. Multiple by 0.6 for ASD.

L = 30 L/B = 1.50
B = 20 h/L = 0.52

Design Pressure (psf)

	z (ft)	q_z (psf)	Cp	qGC_p	(+GCpi)	(-GCpi)
Windward Wall	15	22.79	0.80	15.50	**11.34**	**19.65**
	20	24.74	0.80	16.83	**12.67**	**20.98**
	25	26.37	0.80	17.93	**13.78**	**22.09**
	30	27.78	0.80	18.89	**14.74**	**23.05**
Leeward Wall	15.67	23.08	-0.40	-7.85	**-12.00**	**-3.69**
Side Wall	15.67	23.08	-0.70	-13.73	**-17.88**	**-9.58**
Windward Roof (Positive)	15.67	23.08	-0.06	-1.21	**-5.37**	**2.94**
Windward Roof (Negative)	15.67	23.08	-0.51	-9.95	**-14.10**	**-5.80**
Leeward Roof	15.67	23.08	-0.57	-11.18	**-15.34**	**-7.03**
Ridge Parallel Roof	(0 to h/2)	23.08	-0.92	-18.00	**-22.16**	**-13.85**
	(h/2 to h)	23.08	-0.89	-17.48	**-21.63**	**-13.33**
	(h to 2h)	23.08	-0.51	-9.98	**-14.14**	**-5.83**
	(>h2)	23.08	-0.32	-6.23	**-10.39**	**-2.08**

$p = qGC_p$

Overhangs:	z (ft)	q_z (psf)	Cp	qGC_p
Windward Overhang	14.000	22.79	0.80	**15.50**

WIND (C&C)

Wind Analysis Method		Part 1: Low Rise Buildings		
Basic Wind Speed (ultimate)		135.00	MPH	
Topography Factor	K_{zt} =	1.00		ASCE 7-10 Fig. 26.8-1
Directionality Factor	K_d =	0.85		ASCE 7-10 Fig. 26.6-1
Internal Pressure Coefficients	(GC_{pi}) =	0.18	-0.18	ASCE 7-10 Table 26.11-1
Roof Pitch		4.00	:12	18.43 DEG
Roof Eve Height		14.000	FT	
Peak Roof Height		17.333	FT	α = 7
Mean Roof Height		15.667	FT	z_g = 1200
Terrain Exp. Category		B		

Velocity Pressure

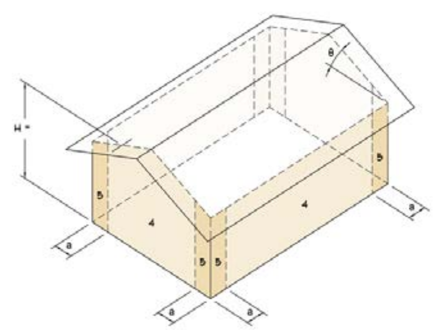
$q_z = .00256 K_z K_{zt} K_d V^2$

Height (ft)			K_z	q_z
h =	15.67	FT	0.701	**27.78**

Wall Components

$p = q_h (GC_p - GC_{pi})$

Component	Span (ft.)	Width (ft.)	Trib. Area	Eff. Area
Girts	10	2.00	20.00	33.33
Panel	8	4	32.00	32.00
$A \le 10$ ft^2	-	-	-	10.00
$A = 20$ ft^2	-	-	-	20.00
$A = 50$ ft^2	-	-	-	50.00
$A = 100$ ft^2	-	-	-	100.00
$A = 200$ ft^2	-	-	-	200.00
$A \ge 500$ ft^2	-	-	-	500.00



Wall Coeficients taken from ASCE 7-10 Fig. 30.4-1

Wall Coefficients

Component	Eff. Area	Zone 4 Pos	Zone 4 Neg	Zone 5 Pos	Zone 5 Neg
Girts	33.33	0.91	-1.01	0.91	-1.22
Panel	32.00	0.91	-1.01	0.91	-1.22
$A \le 10$ ft^2	10.00	1.00	-1.10	1.00	-1.40
$A = 20$ ft^2	20.00	0.95	-1.05	0.95	-1.29
$A = 50$ ft^2	50.00	0.88	-0.98	0.88	-1.15
$A = 100$ ft^2	100.00	0.82	-0.92	0.82	-1.05
$A = 200$ ft^2	200.00	0.77	-0.87	0.77	-0.94
$A \ge 500$ ft^2	500.00	0.70	-0.80	0.70	-0.80

Wall Design Pressures (psf)

Component	Eff. Area	Zone 4 Pos	Zone 4 Neg	Zone 5 Pos	Zone 5 Neg
Girts	33.33	**30.22**	**-33.00**	**30.22**	**-38.77**
Panel	32.00	**30.31**	**-33.08**	**30.31**	**-38.94**
$A \le 10$ ft^2	10.00	32.78	-35.56	32.78	-43.90
$A = 20$ ft^2	20.00	31.31	-34.09	31.31	-40.94
$A = 50$ ft^2	50.00	29.36	-32.13	29.36	-37.04
$A = 100$ ft^2	100.00	27.88	-30.66	27.88	-34.09
$A = 200$ ft^2	200.00	26.40	-29.18	26.40	-31.13
$A \ge 500$ ft^2	500.00	24.45	-27.23	24.45	-27.23

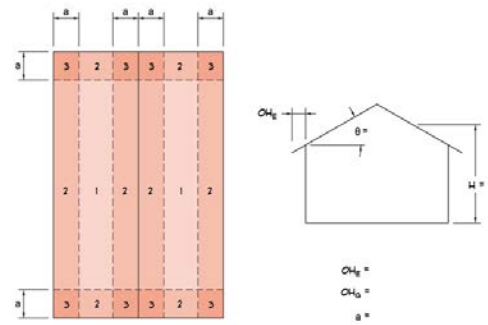
Note: Pressures are limit state design pressures for strength design. Multiple by 0.6 for ASD.

Min. Pressure: The design wind pressure for C&C shall not be less than 16 psf acting in either direction normal to the surface.

Roof Components

$p = q_h(GC_p - GC_{pi})$

Component	Span (ft.)	Width (ft.)	Trib. Area	Eff. Area
Truss/Rafter	20	2	40.00	133.33
Panel	8	4	32.00	32.00
A ≤ 10 ft^2	-	-	-	10.00
A = 20 ft^2	-	-	-	20.00
A = 50 ft^2	-	-	-	50.00
A ≥ 100 ft^2	-	-	-	100.00



Roof Coeficients taken from ASCE 7-10 Fig. 30.4-2B and Fig. 30.4-2C

Roof Coefficients

Component	Eff. Area	Zone 1 Pos	Zone1 Neg	Zone 2 Pos	Zone 2 Neg	Zone 3 Pos	Zone 3 Neg
Truss/Rafter	133.33	0.30	-0.80	0.30	-1.20	0.30	-2.00
Panel	32.00	0.40	-0.85	0.40	-1.45	0.40	-2.30
A ≤ 10 ft^2	10.00	0.50	-0.90	0.50	-1.70	0.50	-2.60
A = 20 ft^2	20.00	0.44	-0.87	0.44	-1.55	0.44	-2.42
A = 50 ft^2	50.00	0.36	-0.83	0.36	-1.35	0.36	-2.18
A ≥ 100 ft^2	100.00	0.30	-0.80	0.30	-1.20	0.30	-2.00

Roof Design Pressures (psf)

Component	Eff. Area	Zone 1 Pos	Zone1 Neg	Zone 2 Pos	Zone 2 Neg	Zone 3 Pos	Zone 3 Neg
Truss/Rafter	133.33	**13.34**	**-27.23**	**13.34**	**-38.34**	**13.34**	**-60.57**
Panel	32.00	**16.09**	**-28.60**	**16.09**	**-45.22**	**16.09**	**-68.82**
A ≤ 10 ft^2	10.00	18.89	-30.01	18.89	-52.23	18.89	-77.24
A = 20 ft^2	20.00	17.22	-29.17	17.22	-48.05	17.22	-72.22
A = 50 ft^2	50.00	15.01	-28.06	15.01	-42.52	15.01	-65.59
A = 100 ft^2	100.00	13.34	-27.23	13.34	-38.34	13.34	-60.57

Roof Coefficients **(Overhang)**

Component	Eff. Area	Zone 1 Pos	Zone1 Neg	Zone 2 Pos	Zone 2 Neg	Zone 3 Pos	Zone 3 Neg
Truss/Rafter	133.33	0.30	-0.80	0.30	-2.20	0.30	-2.50
Panel	32.00	0.40	-0.85	0.40	-2.20	0.40	-3.09
A ≤ 10 ft^2	10.00	0.50	-0.90	0.50	-2.20	0.50	-3.70
A = 20 ft^2	20.00	0.44	-0.87	0.44	-2.20	0.44	-3.34
A = 50 ft^2	50.00	0.36	-0.83	0.36	-2.20	0.36	-2.86
A ≥ 100 ft^2	100.00	0.30	-0.80	0.30	-2.20	0.30	-2.50

Roof Design Pressures **(Overhang)** (psf)

Component	Eff. Area	Zone 1 Pos	Zone1 Neg	Zone 2 Pos	Zone 2 Neg	Zone 3 Pos	Zone 3 Neg
Truss/Rafter	133.33	**13.34**	**-27.23**	**13.34**	**-66.13**	**13.34**	**-74.46**
Panel	32.00	**16.09**	**-28.60**	**16.09**	**-66.13**	**16.09**	**-90.96**
A ≤ 10 ft^2	10.00	18.89	-30.01	18.89	-66.13	18.89	-107.80
A = 20 ft^2	20.00	17.22	-29.17	17.22	-66.13	17.22	-97.76
A = 50 ft^2	50.00	15.01	-28.06	15.01	-66.13	15.01	-84.50
A = 100 ft^2	100.00	13.34	-27.23	13.34	-66.13	13.34	-74.46

Width of Zones 2,3 and 5

smaller of:	0.1 x	20.00 =	2.00 ft			
	0.4 x	15.67 =	6.27 ft			
not less than:	0.04 x	20.00 =	0.80 ft			
		or	3 ft		**(controls)**	a = 3 ft

Note: Pressures are limit state design pressures for strength design. Multiple by 0.6 for ASD.

Min. Pressure: The design wind pressure for C&C shall not be less than 16 psf acting in either direction normal to the surface.