MEDEEK ENGINEERING INC.
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ENGINEERING REPORT

STRUCTURAL REVIEW

August 5, 2015

JOB NUMBER: **2015-035**

PLAN NUMBER: BAXTER MINI-STORAGE

CUSTOMER: MARK BAXTER

LOCATION: 3019 OCEAN BEACH RD. PACIFIC BEACH WA 98571

Engineer's seal applies to this entire calculation packet. This packet is void if engineer's seal is not an original and signature is not signed in blue ink.

Engineer: Nathaniel P. Wilkerson

This engineering report is valid only for the building located at 3019 Ocean Beach Rd., Pacific Beach WA 98571.

Job#: **2015-035**

ENGINEERING REPORT: STRUCTURAL REVIEW

Customer: Mark Baxter
Location: 3019 Ocean Beach Rd. Pacific Beach WA 98571
Engr: Nathaniel P. Wilkerson
Date: 29-Jul-15

CODES

ICC International Building Code IBC 2012	American Concrete Institute ACI 318-11
Minimum Design Loads for Buildings ASCE7-10	AWC NDS 2012

DESIGN CRITERIA SUMMARY

Ground Snow Load	25.0	PSF
Frost Line Depth	12.0	IN
Occupancy Classification	S-1	
Risk Category	II	
Snow Importance Factor (I_s)	1.0	
Wind Speed (ultimate)	135.0	MPH
Terrain Exp. Category	C	
Wind Importance Factor (I_w)	1	
Wind Factor in Load Combinations (ASD)	0.6	
Site Class	D	Stiff Soil
Seismic Design Category (SDC)	D	
Seismic Factor in Load Combinations (ASD)	0.7	
Seismic Importance Factor (I_e)	1.0	
Construction Type	V-B	
Soil Bearing Capacity	1500.0	PSF

LOADS

Floor Dead Load	10.0	PSF	
Floor Live Load	40.0	PSF	
Roof TC Dead Load	7.0	PSF	
Roof BC Dead Load	5.0	PSF	
Ceiling Dead Load (Gypsum)	5.0	PSF	
Roof Live Load (Construction)	20.0	PSF	
Roof Snow Load (P_s) [See Snow Load Report]	19.3	PSF	(governs)
Stair Live Load	40.0	PSF	
Deck Live Load	50.0	PSF	

BUILDING DATA

Roof Pitch	4.00	:12
Roof Eve Height	8.500	FT
Peak Roof Height	16.167	FT
Mean Roof Height	12.334	FT
Building Length (L)	83.625	FT
Building Width (B)	42.875	FT
Latitude	47.2026	N
Longitude	124.1708	W
Elevation:	55.0	FT

SEISMIC

S_{DS}		0.984	g
S_{D1}		0.735	g
S_{MS}		1.476	g
S_{M1}		1.103	g
S_S		1.476	g
S_1		0.735	g
F_a		1.000	
F_v		1.500	
Roof Diaphragm Height (h_n)*		12.35	FT
Fundamental Period (T_a)	$T_a = C_t h_n^x$ =	0.132	sec.
T_0		0.149	sec.
T_S		0.747	sec.
T_L (Fig. 22-12)		16.0	sec.
Response Modification Factor (R)		6.5	WSP SWL
Response Modification Factor (R)		2	GYP SWL
Deflection Amplification Factor (C_d)		4	WSP SWL
Overstrength factor (Ω_0)		3	WSP SWL
Redundancy Factor (ρ)		1.3	(SDC D)
Seismic Response Coef.(C_s)	$C_s = \dfrac{S_{DS}}{\left(\dfrac{R}{I_e}\right)}$ =	**0.151**	
Max. Seismic Response Coef.(C_{smax}) (for $T_a \le T_L$)	$C_s = \dfrac{S_{D1}}{T_a\left(\dfrac{R}{I_e}\right)}$ =	0.858	

Min. Seismic Response Coef.(C_{smin})

	$C_s = 0.044 S_{DS} I_e \ge 0.01$ =	0.039	
if $S_1 \ge 0.6$g:	$C_s = \dfrac{0.5 S_1}{\left(\dfrac{R}{I_e}\right)}$ =	0.057	

*For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

IRC Seismic Design Categories TM

Use our **IRC Seismic Design Categories** map to easily obtain the seismic design category (Figure R301.2(2) of IRC 2012) for any location in the contiguous United States, Puerto Rico and Alaska. You can click on the map below to determine the seismic design category for that location.

The seismic design category (SDC) is calculated based on the design spectral response acceleration (S_{ds} at Site Class = D, Risk Cat. = II), provided by the USGS Seismic API.

Street: 3019 Ocean Beach Rd. City: Pacific Beach State: Washington Zip: 98571 Find

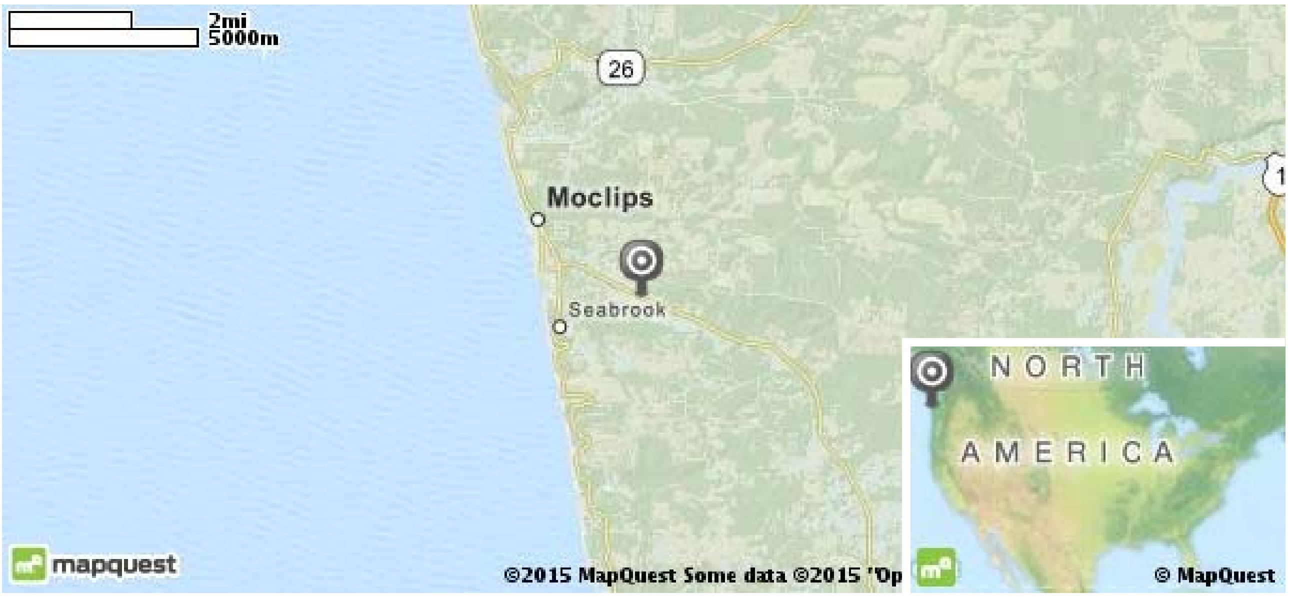
* Seismic Design Categories calculated from USGS Seismic API data. Local codes and ammendments may govern, verify with local building department or jurisdiction.

If you need to gather seismic data programmatically, please consider our *API Service*.
If you have any questions or concerns please call us at 1-425-741-5555.

Design Maps Summary Report

User–Specified Input

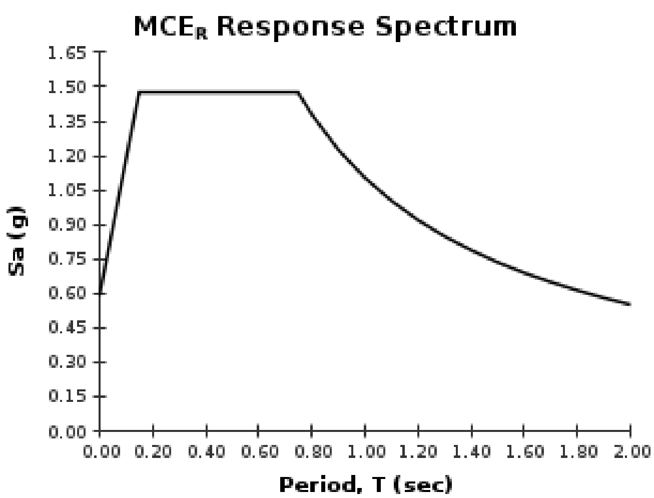
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	47.2026°N, 124.1708°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

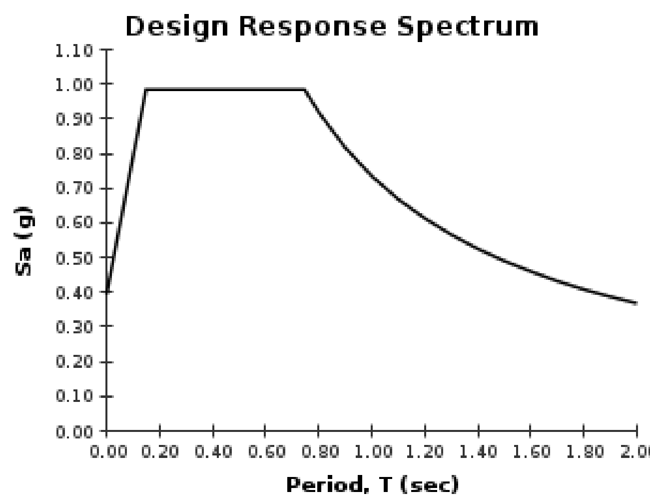


USGS–Provided Output

S_S =	1.476 g	S_{MS} =	1.476 g	S_{DS} =	0.984 g
S_1 =	0.735 g	S_{M1} =	1.103 g	S_{D1} =	0.735 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	4 /12
Risk Category:	II
Eave-to-Ridge (W):	22.7708 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	truss
Spacing:	24 in. o/c
Overhang:	16 in.

2. Design Loads

Top Chord Dead Load: 7 psf

Bottom Chord Dead Load: 10 psf

SF (Slope Factor) = 1/Cosine(Φ) = 1.05 (Dead loads specified on a projected horizontal basis take into account the effect of the pitch via a slope factor.)

Adj. TCDL (TCDL x SF): 7.4 psf

3. Design Assumptions

Code Standard: ASCE 7-10

Number of Plies: 1 PLY

Bottom Chord Pitch: 0 /12

4. Snow Load Calculations

Calculate flat roof snow load p_f using the following equation:

$p_f = 0.7C_eC_tI_sp_g$

where:

p_f = Flat Roof Snow Load in psf

C_e = 1.00 = Exposure Factor, as determined by ASCE 7-10 Table 7-2 (Terrain Cat. C, Exp. Partially Exposed)

C_t = 1.10 = Thermal Factor, as determined by ASCE 7-10 Table 7-3

I_s = 1.00 = Importance Factor, as determined by ASCE 7-10 Table 1.5-2 (Risk Cat. II)

p_g = 25.0 psf = Ground Snow Load in psf

$p_f = 0.7C_eC_tI_sp_g = 0.7(1.00)(1.10)(1.00)(25.0) = 19.3$ psf

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A minimum roof snow load, p_m shall apply to monoslope, hip and gable roofs with slopes less than 15 degrees using the following equations:

Where p_g is 20 psf or less: $p_m = I_s p_g$
Where p_g exceeds 20 psf: $p_m = I_s(20)$

Roof slope is greater than 15 degrees, the minimum roof snow load, p_m, does not apply.

For locations where p_g is 20 psf or less, but not zero, all roofs with slopes (in degrees) less than W/50 with W in feet shall included a 5 psf rain-on-snow surcharge load. This additional load applies only to the sloped roof (balanced) load case and need not be used in combination with drift, sliding, unbalanced, minimum, or partial loads.

Roof slope in degrees (18.43°) is greater than W/50 = 0.5, the 5.0 psf rain-on-snow surcharge load does not apply.

Calculate sloped roof snow load p_s using the following equation:

$p_s = C_s p_f$

where:

p_s = Sloped Roof Snow Load in psf
C_s = 1.00 = Roof Slope Factor, as determined by ASCE 7-10 Sec. 7.4.1-7.4.4 and Figure 7-2
p_f = Flat Roof Snow Load in psf

Roof surface (Asphalt Shingles) is considered a "non-slippery" roof. For a C_t = 1.10 the roof slope factor C_s is given by the solid line of ASCE 7-10 Figure 7-2b.

$p_s = C_s p_f = (1.00)(19.3) = 19.3$ psf

Calculate unbalanced snow load for hip and gable roofs as shown in ASCE 7-10 Figure 7-5.
Unbalanced snow loads are required for roof pitches between 1/2 on 12 to 7 on 12.
Using the following equations:

$\gamma = 0.13p_g + 14$ (snow density)
$h_d = .43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5$ (drift height)
$l_d = \frac{8}{3} h_d \sqrt{S}$ (width of drift surcharge)
$p_d = h_d \gamma / \sqrt{S}$ (drift surcharge snow load)

where:

γ = Snow density in pcf, not to exceed 30 pcf.
h_d = Drift height in feet, as determined by eqn. or ASCE 7-10 Fig. 7-9.
l_u = W = Ridge to eave distance in feet, windward side of roof.
S = 12/Roof Pitch
l_d = Width of drift surcharge in feet.
p_d = Drift Surcharge Snow Load in psf

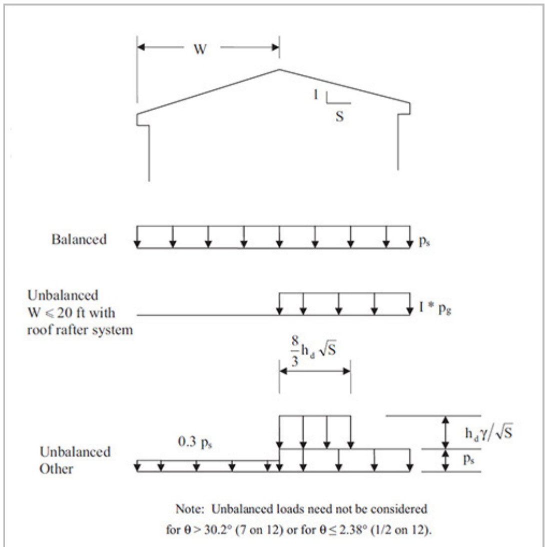

FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

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$p_{windward} = 0.3p_s = (0.3)(19.3) = 5.8$ psf
$p_{leeward} = p_s = 19.3$ psf

$\gamma = 0.13(25.0) + 14 = 17.25$ pcf

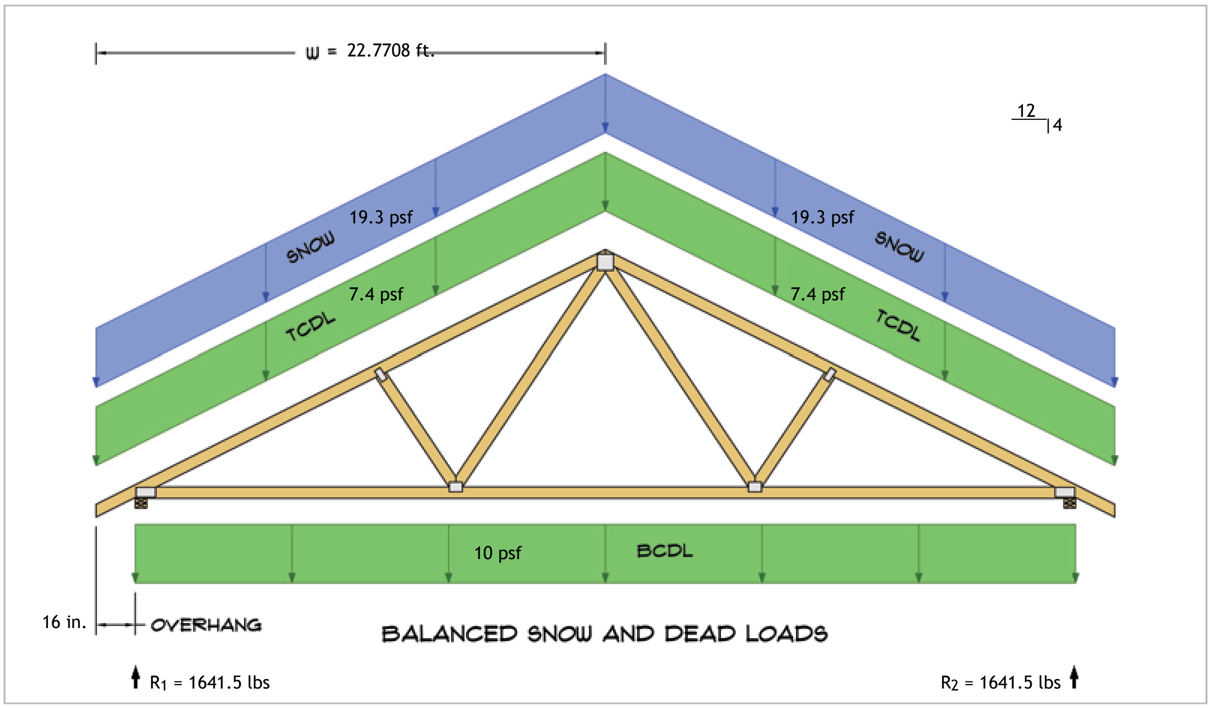
$$h_d = .43\sqrt[3]{22.7708}\sqrt[4]{25.0+10} - 1.5 = 1.46 \text{ ft.}$$

$$l_d = \frac{8}{3} \times 1.46 \times \sqrt{12/4} = 6.76 \text{ ft.}$$

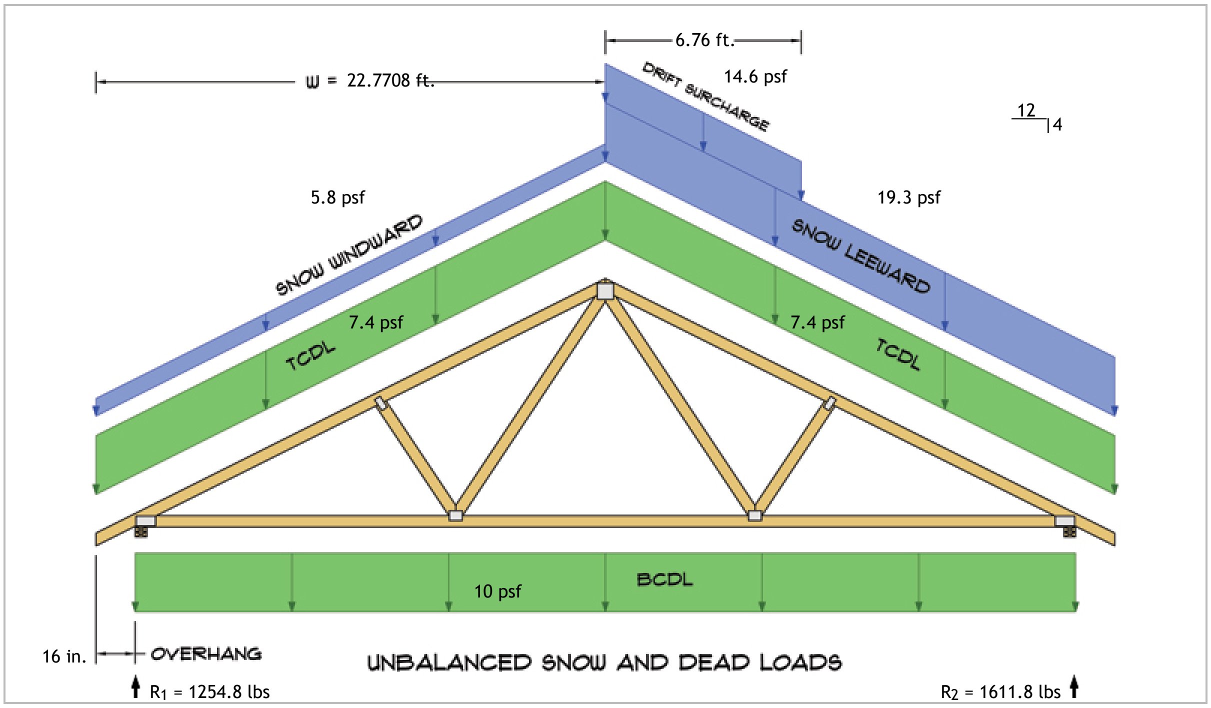
$$p_d = \frac{1.46 \times 17.25}{\sqrt{12/4}} = 14.6 \text{ psf}$$

On warm roofs apply a distributed $2p_f$ snow load on all overhanging portions as per ASCE 7-10 section 7.4.5.
No other loads except dead loads shall be present on the roof when this uniformly distributed load is applied.

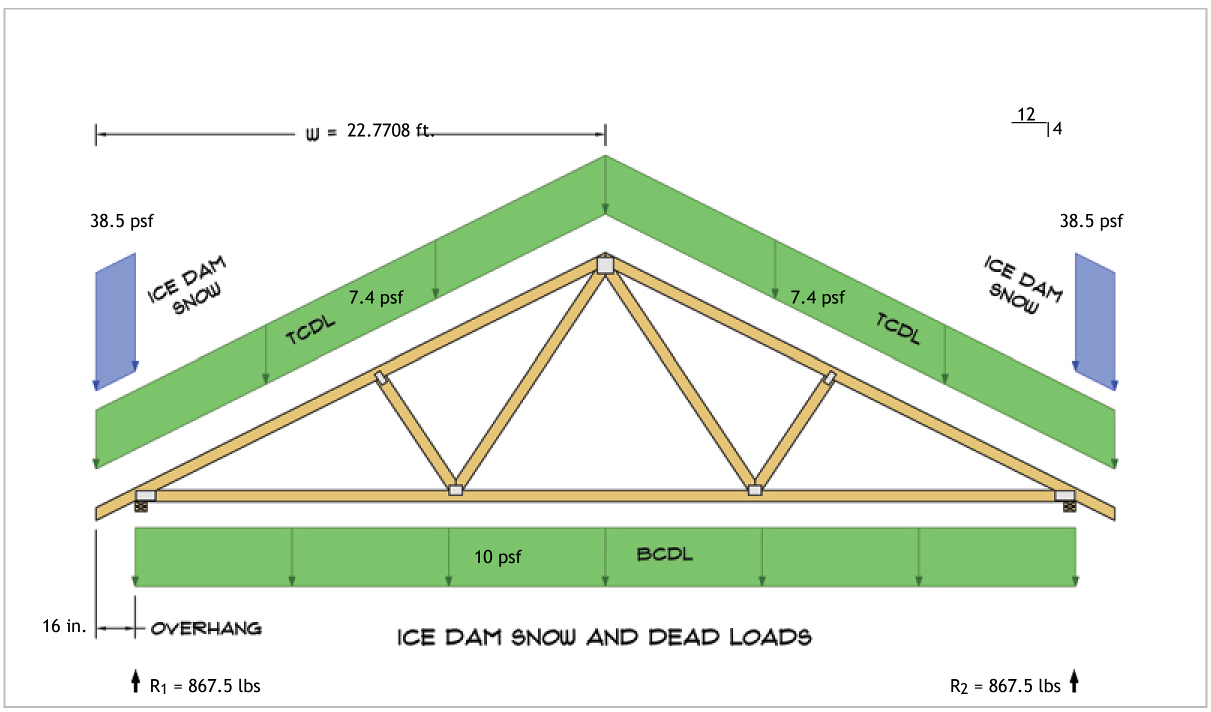
$2p_f = (2)(19.3) = 38.5$ psf



$R_1 = D + S = 764.8$ lbs $+ 876.7$ lbs
$R_2 = D + S = 764.8$ lbs $+ 876.7$ lbs



R_1 = D + S = 764.8 lbs + 490.0 lbs
R_2 = D + S = 764.8 lbs + 847.0 lbs



R_1 = D + S = 764.8 lbs + 102.7 lbs
R_2 = D + S = 764.8 lbs + 102.7 lbs

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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		8.500	FT	
Peak Roof Height		16.167	FT	α = 9.5
Mean Roof Height		12.334	FT	z_g = 900
Terrain Exp. Category		C		

Velocity Pressures

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

Height (ft)			K_z	q_z
h_e =	8.50	FT	0.849	33.66
h =	12.33	FT	0.849	**33.66**
z =	15	FT	0.849	33.66
z =	20	FT	0.902	35.77
z =	25	FT	0.945	37.49
z =	30	FT	0.982	38.95

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 = 42.875 L/B = 0.51
B = 83.625 h/L = 0.29

	z (ft)	q_z (psf)	Cp	qGC_p	Design Pressure (psf) (+GCpi)	(-GCpi)
Windward Wall	15	33.66	0.80	22.89	**16.83**	**28.95**
	20	35.77	0.80	24.32	**18.26**	**30.38**
	25	37.49	0.80	25.49	**19.43**	**31.55**
	30	38.95	0.80	26.49	**20.43**	**32.55**
Leeward Wall	12.33	33.66	-0.50	-14.31	**-20.37**	**-8.25**
Side Wall	12.33	33.66	-0.70	-20.03	**-26.09**	**-13.97**
Windward Roof (Positive)	12.33	33.66	0.11	3.10	**-2.96**	**9.16**
Windward Roof (Negative)	12.33	33.66	-0.38	-10.94	**-17.00**	**-4.88**
Leeward Roof	12.33	33.66	-0.57	-16.27	**-22.33**	**-10.21**
Ridge Parallel Roof	(0 to h/2)	33.66	-0.90	-25.75	**-31.81**	**-19.69**
	(h/2 to h)	33.66	-0.90	-25.75	**-31.81**	**-19.69**
	(h to 2h)	33.66	-0.50	-14.31	**-20.37**	**-8.25**
	(>h2)	33.66	-0.30	-8.58	**-14.64**	**-2.52**

Longitudinal Direction:

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

L = 83.625 L/B = 1.95
B = 42.875 h/L = 0.15

	z (ft)	q_z (psf)	Cp	qGC_p	Design Pressure (psf) (+GCpi)	(-GCpi)
Windward Wall	15	33.66	0.80	22.89	**16.83**	**28.95**
	20	35.77	0.80	24.32	**18.26**	**30.38**
	25	37.49	0.80	25.49	**19.43**	**31.55**
	30	38.95	0.80	26.49	**20.43**	**32.55**
Leeward Wall	12.33	33.66	-0.31	-8.87	**-14.93**	**-2.81**
Side Wall	12.33	33.66	-0.70	-20.03	**-26.09**	**-13.97**
Windward Roof (Positive)	12.33	33.66	0.14	3.93	**-2.13**	**9.99**
Windward Roof (Negative)	12.33	33.66	-0.36	-10.38	**-16.44**	**-4.32**
Leeward Roof	12.33	33.66	-0.57	-16.27	**-22.33**	**-10.21**
Ridge Parallel Roof	(0 to h/2)	33.66	-0.90	-25.75	**-31.81**	**-19.69**
	(h/2 to h)	33.66	-0.90	-25.75	**-31.81**	**-19.69**
	(h to 2h)	33.66	-0.50	-14.31	**-20.37**	**-8.25**
	(>h2)	33.66	-0.30	-8.58	**-14.64**	**-2.52**

$p = qGC_p$

Overhangs:	z (ft)	q_z (psf)	Cp	qGC_p
Windward Overhang	8.500	33.66	0.80	**22.89**

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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		8.500	FT		
Peak Roof Height		16.167	FT	α =	9.5
Mean Roof Height		12.334	FT	zg =	900
Terrain Exp. Category		C			

Velocity Pressure

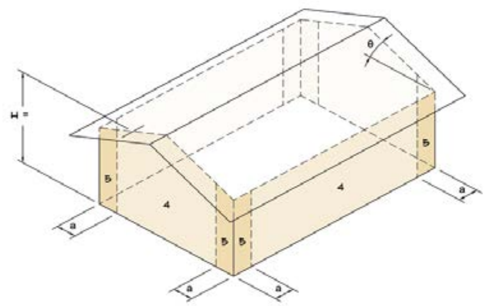
$q_z = .00256 K_z K_{zt} K_d V^2$

Height (ft)			K_z	q_z
h =	12.33	FT	0.849	**33.66**

Wall Components

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

Component	Span Length (ft.)	Width (ft.)	Trib. Area	Eff. Area
Stud	8	1.33	10.64	21.33
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
A = 200 ft^2	-	-	-	200.00
A ≥ 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
Stud	21.33	0.94	-1.04	0.94	-1.28
Panel	32.00	0.91	-1.01	0.91	-1.22
A ≤ 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 ≥ 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
Stud	21.33	**37.77**	**-41.13**	**37.77**	**-49.28**
Panel	32.00	**36.72**	**-40.09**	**36.72**	**-47.18**
A ≤ 10 ft^2	10.00	39.72	-43.09	39.72	-53.19
A = 20 ft^2	20.00	37.93	-41.30	37.93	-49.61
A = 50 ft^2	50.00	35.57	-38.94	35.57	-44.88
A = 100 ft^2	100.00	33.78	-37.15	33.78	-41.30
A = 200 ft^2	200.00	31.99	-35.36	31.99	-37.72
A ≥ 500 ft^2	500.00	29.62	-32.99	29.62	-32.99

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.

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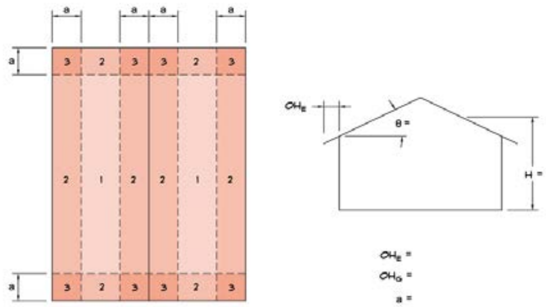
$p = q_h(GC_p - GC_{pi})$

Roof Components

Component	Span Length (ft.)	Width (ft.)	Trib. Area	Eff. Area
Truss/Rafter	42.875	2	85.75	612.76
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	612.76	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	612.76	**16.16**	**-32.99**	**16.16**	**-46.46**	**16.16**	**-73.39**
Panel	32.00	**19.49**	**-34.66**	**19.49**	**-54.79**	**19.49**	**-83.38**
A ≤ 10 ft^2	10.00	22.89	-36.36	22.89	-63.29	22.89	-93.59
A = 20 ft^2	20.00	20.87	-35.34	20.87	-58.22	20.87	-87.51
A = 50 ft^2	50.00	18.19	-34.00	18.19	-51.52	18.19	-79.47
A = 100 ft^2	100.00	16.16	-32.99	16.16	-46.46	16.16	-73.39

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	612.76	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	612.76	**16.16**	**-32.99**	**16.16**	**-80.12**	**16.16**	**-90.22**
Panel	32.00	**19.49**	**-34.66**	**19.49**	**-80.12**	**19.49**	**-110.21**
A ≤ 10 ft^2	10.00	22.89	-36.36	22.89	-80.12	22.89	-130.62
A = 20 ft^2	20.00	20.87	-35.34	20.87	-80.12	20.87	-118.46
A = 50 ft^2	50.00	18.19	-34.00	18.19	-80.12	18.19	-102.38
A = 100 ft^2	100.00	16.16	-32.99	16.16	-80.12	16.16	-90.22

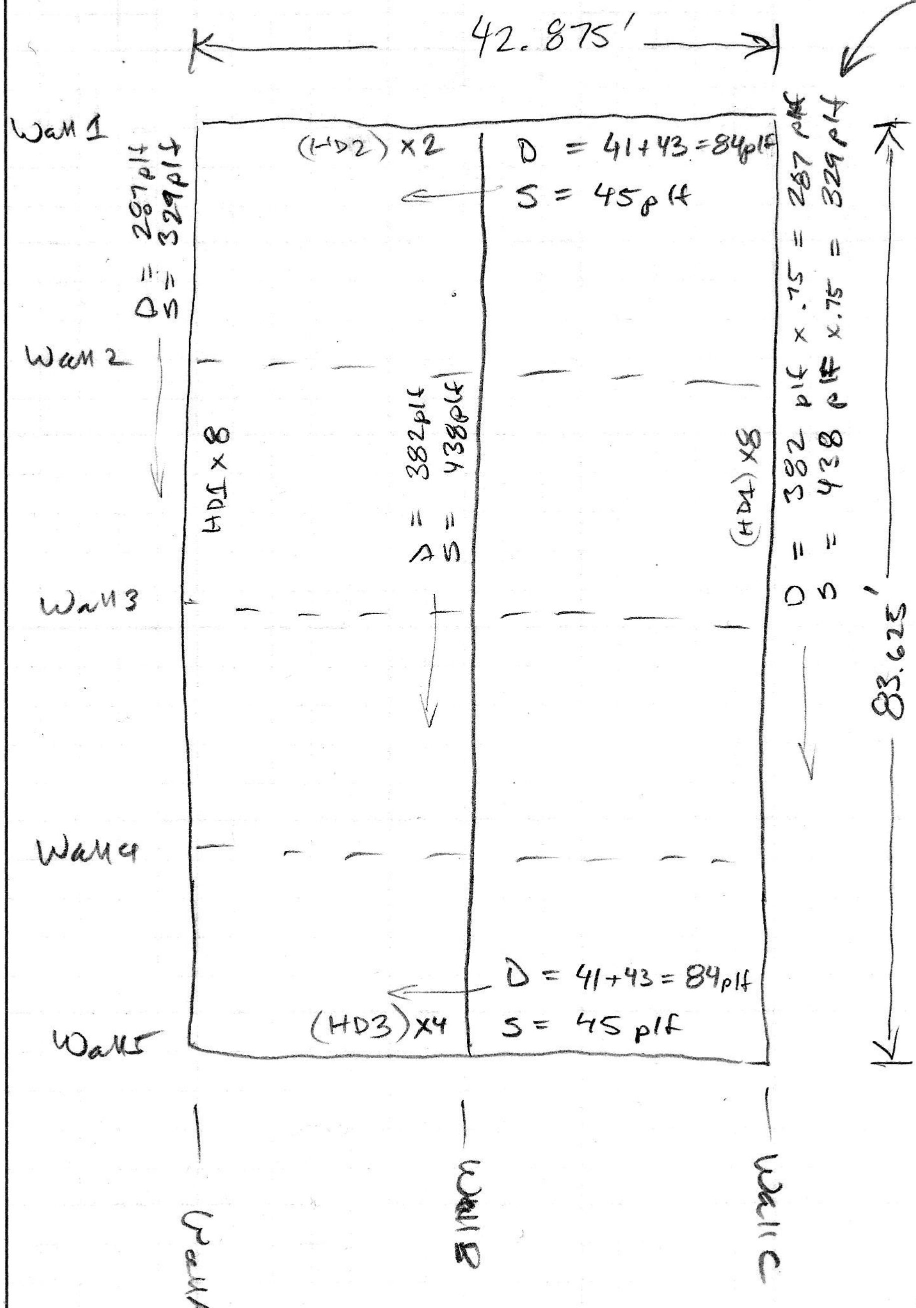
Width of Zones 2,3 and 5

smaller of:	0.1 x	42.88 =	4.29	ft	**(controls)**	a = 4.2875 ft
	0.4 x	12.33 =	4.93	ft		
not less than:	0.04 x	42.88 =	1.72	ft		
		or	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.

Bearing Wall Plan



conservative @ 75% of load without center bearing wall

Loads

$S = 19.3\ psf$

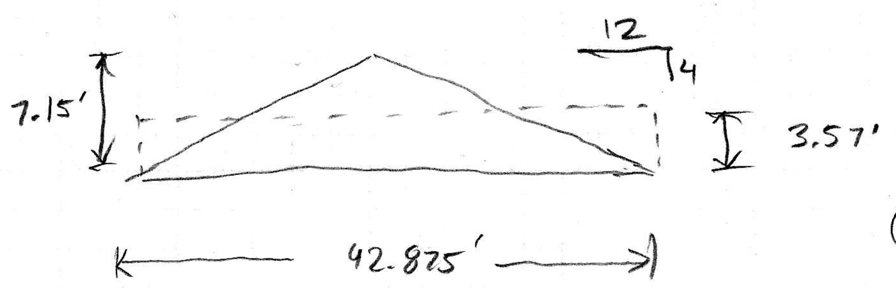
$D = 17.4\ psf$

@ Gable Overhang

$L_{trib} = 1 + 1.33$

$= 2.33'$

Gable Ends



Wall Dead Weight = 12 psf

(12 psf) (3.57') = 43 plf

Footings and Foundations

Check footing soil pressure at highest (vertically) loaded section of wall excluding point loads.
From previous sections and by inspection one of the most critically loaded walls is at Wall Line A.

(plf)	Dead Load	Floor Live	Roof Live	Roof Snow
Roof	287	0	0	329
Wall	96	0	0	0
Floor	0	0	0	0
Stemwall	0	0	0	0
Totals	383	0	0	329

Wall DL =	12	psf
Wall Hgt. =	8	ft
$trib_{floor}$ =	0.0	ft
Stem Width =	0	in
Stem Hgt. =	0	in
ρ_{conc} =	150	pcf

Assume footing width and soil bearing:

Footing Width =	12	in
Soil Bearing Pressure =	1500	psf
ρ_{soil} =	100	pcf
Soil Depth Above Ftg.	0	in
Footing Depth =	18	in

Load Cases from ASCE 7-10:

2.) D + L =	383	plf	
3.) D + (Lr or S) =	712	plf	(governs)
4.) D + .75L + .75(Lr or S) =	630	plf	

Eff. Allowable SBP — Q_e = 1,275 psf

Req. Soil Bearing Pressure = **712** psf < 1,275 psf ⟶ **OK**

Use 12" x 18" x cont. turned down thickened edge slab footing with (2) #4 bars cont. horizontal top and bottom

From previous sections and by inspection one of the most critically loaded walls is at Wall Line B.

(plf)	Dead Load	Floor Live	Roof Live	Roof Snow
Roof	382	0	0	438
Wall	96	0	0	0
Floor	0	0	0	0
Stemwall	0	0	0	0
Totals	478	0	0	438

Wall DL =	12	psf
Wall Hgt. =	8	ft
$trib_{floor}$ =	0.0	ft
Stem Width =	0	in
Stem Hgt. =	0	in
ρ_{conc} =	150	pcf

Assume footing width and soil bearing:

Footing Width =	16	in
Soil Bearing Pressure =	1500	psf
ρ_{soil} =	100	pcf
Soil Depth Above Ftg.	0	in
Footing Depth =	12	in

Load Cases from ASCE 7-10:

2.) D + L =	478	plf	
3.) D + (Lr or S) =	916	plf	(governs)
4.) D + .75L + .75(Lr or S) =	807	plf	

Eff. Allowable SBP — Q_e = 1,350 psf

Req. Soil Bearing Pressure = **687** psf < 1,350 psf ⟶ **OK**

Use 16" x 12" x cont. thickened slab footing with (2) #4 bars cont. horizontal.

Pad Footing at HD1 Trimmer:

Stemwall Height:	16	in
Stemwall Footing Width:	12	in
Assume square footing:	0	in

Pmax = 2714 lbs

Req. Bearing (Stemwall Ftg. Only) = P/A =	2,714	/	2.00	=	1,357	psf
Req. Bearing (Sq. Ftg. Only) = P/A =	2,714	/	0.00	=	N/A	
Req. Bearing (Combined) = P/A =	2,714	/	2.00	=	1,357	psf

Req Soil Bearing = 1,357 psf < 1,500 psf ⟶ **OK**

01: FLOOR1			
Member Name	**Results**	**Current Solution**	**Comments**
HD1	Passed	3 Piece(s) 2 x 10 Douglas Fir-Larch No. 2	
HD2	Passed	3 Piece(s) 2 x 10 Douglas Fir-Larch No. 2	
HD3	Passed	2 Piece(s) 2 x 10 Douglas Fir-Larch No. 2	

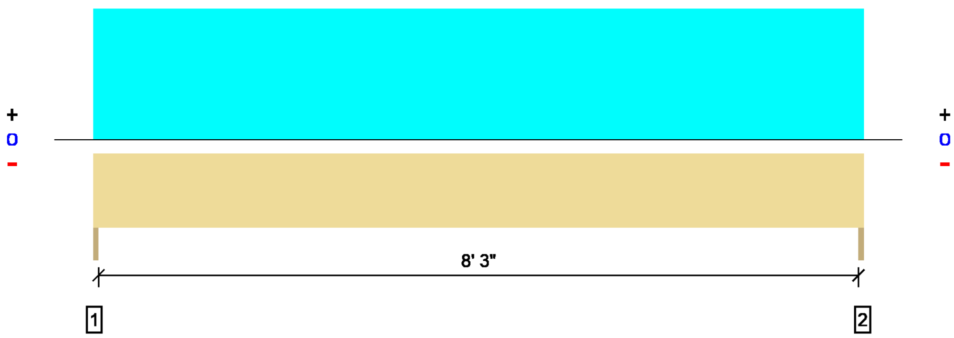
Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 741-5555 nathan@medeek.com	Job#: 2015-035 Mark Baxter Pacific Beach WA 98571

MEMBER REPORT *FLOOR1, HD1*

PASSED

3 piece(s) 2 x 10 Douglas Fir-Larch No. 2

Overall Length: 8' 6"



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2714 @ 0	4219 (1.50")	Passed (64%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2142 @ 10 3/4"	5744	Passed (37%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	5767 @ 4' 3"	6088	Passed (95%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.081 @ 4' 3"	0.283	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.158 @ 4' 3"	0.425	Passed (L/646)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 7' 4 9/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS 2005 methodology.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Total	Accessories
1 - Trimmer - DF	1.50"	1.50"	1.50"	1316	1398	2714	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1316	1398	2714	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 8' 6"	N/A	287.0	329.0	ROOF
2 - Uniform (PSF)	0 to 8' 6"	1'	12.0	-	EXT. WAll

Member Notes

8x7 Garage Door

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by Nathaniel P. Wilkerson PE

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
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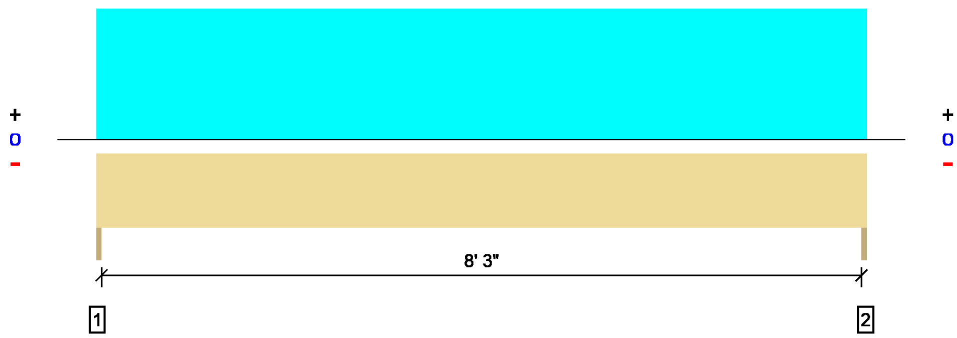
MEMBER REPORT *FLOOR1, HD2*

3 piece(s) 2 x 10 Douglas Fir-Larch No. 2

PASSED

Overall Length: 8' 6"



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1205 @ 0	4219 (1.50")	Passed (29%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	800 @ 10 3/4"	4496	Passed (18%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	2154 @ 4' 3"	4765	Passed (45%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.011 @ 4' 3"	0.283	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.070 @ 4' 3"	0.425	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 8' 6" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS 2005 methodology.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1014	191	1205	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1014	191	1205	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 8' 6"	N/A	84.0	45.0	ROOF
2 - Uniform (PSF)	0 to 8' 6"	12'	12.0	-	EXT. WAll

Member Notes

8x8 Garage Door

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by Nathaniel P. Wilkerson PE

Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 741-5555 nathan@medeek.com	Job#: 2015-035 Mark Baxter Pacific Beach WA 98571

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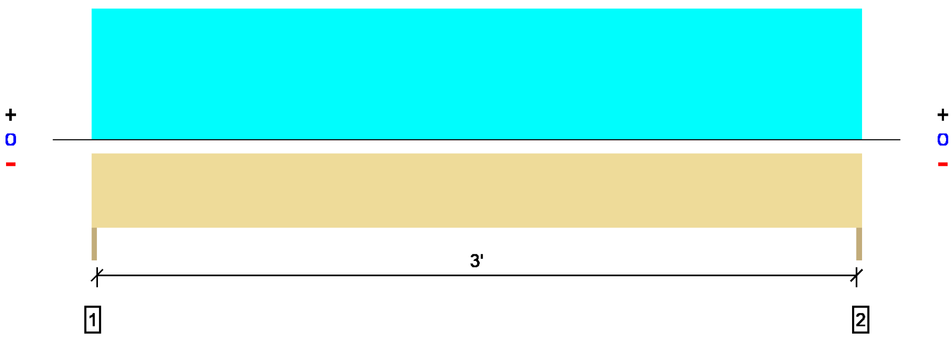
2015-035_HEADERS.4te

FORTE® **MEMBER REPORT** *FLOOR1, HD3* **PASSED**

2 piece(s) 2 x 10 Douglas Fir-Larch No. 2

Overall Length: 3' 3"



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	241 @ 0	2813 (1.50")	Passed (9%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	108 @ 10 3/4"	3830	Passed (3%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	195 @ 1' 7 1/2"	4059	Passed (5%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.000 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.001 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 3' 3" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS 2005 methodology.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	167	73	240	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	167	73	240	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 3' 3"	N/A	84.0	45.0	ROOF
2 - Uniform (PSF)	0 to 3' 3"	1'	12.0	-	EXT. WAll

Member Notes

3068 Door

Weyerhaeuser Notes

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SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 741-5555 nathan@medeek.com	Job#: 2015-035 Mark Baxter Pacific Beach WA 98571

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Job#:	**2015-035**
Location:	WALLB

STUD WALL CALCULATIONS

Stud Width (dy)	1.50	in
Stud Depth (dx)	3.50	in
Stud Length (L)	8.00	ft
Stud Spacing	16	in
Stud Species and Grade	2X4 DF Stud	
Top/Sill Plt. Species	HF	

Design Values

Fb	700	psi
Fc	850	psi
Fc⊥	405	psi
E	1,400,000	psi
Emin	510,000	psi
CF_b	1.10	
CF_c	1.05	
A	5.25	in^2
Sx	3.06	in^3
Ix	5.36	in^4
Ct_c	1.00	
CM_c	1.00	
Ci_c	1.00	

Load Case 1: Gravity Loads Only

ly (unbraced length)	1.0	ft
CD	1.15	(Snow Load)
(le/d)y	8.00	
(le/d)x	27.43	(governs)
E'min	510,000	psi
FcE	557.23	psi
Fc*	1026.38	psi
c	0.80	sawn lumber
FcE/Fc*	0.543	
1 + FcE/Fc*/2c	0.964	
Cp	0.463	
Fc'	475.26	psi
fc	208.25	psi
CSI (axial)	**0.44**	**OK**

Bearing on Stud Wall Plates

lb	1.50	in
Cb	1.00	(conservative)
Fc⊥'	405.00	psi
fc⊥	208.25	psi
CSI (bearing)	**0.51**	**OK**

Deflection

E'	1,400,000	psi
ΔWIND (.42C&C)*	0.11	in
L/d**	**872**	**OK**

*IBC 2015 Sec. 1604.3

**IRC 2015 Sec. 301.7

Vertical Loads

Wall LL (wLL)	438	plf
Wall DL (wDL)	382	plf
Wall DL (wTL)	820	plf
Trib. Length	1.33	ft
Pc	1093.33	lbs

Lateral Loads (Wind MWFRS)

Wind Load (windward wall)	16.00	psf
MWFRS Wind Load ASD	9.60	psf
Wind Atrib	10.67	ft^2
W	102.40	lbs
w	12.80	plf

Lateral Loads (Wind C&C)

Wind Load (Zone 4)	16.00	psf
CC Wind Load ASD	9.60	psf
W	102.40	lbs
w	12.80	plf

Load Case 2: Lateral Loads Only (Wind C&C)

Mmax	102.40	ft-lbs
	1228.80	in-lbs
fbx	401.24	psi
CSI (bending C&C)	**0.28**	**OK**

Load Case 3: Gravity Loads and Lateral Loads

CD	1.60	(Wind/Seismic)
Mmax	102.40	ft-lbs
	1228.80	in-lbs
CL	1.00	
Cr	1.15	@ 16 O/C
Fbx'	1409.95	psi
fbx	401.24	psi
CSI (bending MWFRFS)	**0.28**	**OK**

Combined Stress

(re-evaluate compression values with CD = 1.6)

FcEx	557.23	psi
FcE	557.23	psi
Fc*	1428.00	psi
c	0.80	sawn lumber
FcE/Fc*	0.390	
1 + FcE/Fc*/2c	0.869	
Cp	0.352	
Fc'	502.63	psi

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}'}\right) = \mathbf{0.63}\ \textbf{OK}$$

Load Case: LCMAX

*LCMAX takes 100% of all loads for axial and bending.

Location:	Wall B
Specification:	**Use 2X4 DF Stud Grade @ 16" o/c**

Rev. 1.3.1 - 06/23/2015

Job#:	**2015-035**
Location:	WALLA

STUD WALL CALCULATIONS

Stud Width (dy)	1.50	in
Stud Depth (dx)	5.50	in
Stud Length (L)	8.00	ft
Stud Spacing	16	in
Stud Species and Grade	2X6 DF Stud	
Top/Sill Plt. Species	HF	

Vertical Loads

Wall LL (wLL)	329	plf
Wall DL (wDL)	287	plf
Wall DL (wTL)	616	plf
Trib. Length	1.33	ft
Pc	821.33	lbs

Design Values

Fb	700	psi
Fc	850	psi
Fc⊥	405	psi
E	1,400,000	psi
Emin	510,000	psi
CF_b	1.00	
CF_c	1.00	
A	8.25	in^2
Sx	7.56	in^3
Ix	20.80	in^4
Ct_c	1.00	
CM_c	1.00	
Ci_c	1.00	

Lateral Loads (Wind MWFRS)

Wind Load (windward wall)	28.95	psf
MWFRS Wind Load ASD	17.37	psf
Wind Atrib	10.67	ft^2
W	185.28	lbs
w	23.16	plf

Lateral Loads (Wind C&C)

Wind Load (Zone 4)	41.13	psf
CC Wind Load ASD	24.68	psf
W	263.23	lbs
w	32.90	plf

Load Case 2: Lateral Loads Only (Wind C&C)

Mmax	263.23	ft-lbs
	3158.78	in-lbs
fbx	417.69	psi
CSI (bending C&C)	**0.33**	**OK**

Load Case 1: Gravity Loads Only

ly (unbraced length)	1.0	ft
CD	1.15	(Snow Load)
(le/d)y	8.00	
(le/d)x	17.45	(governs)
E'min	510,000	psi
FcE	1376.02	psi
Fc*	977.50	psi
c	0.80	sawn lumber
FcE/Fc*	1.408	
1 + FcE/Fc*/2c	1.505	
Cp	0.794	
Fc'	776.42	psi
fc	99.56	psi
CSI (axial)	**0.13**	**OK**

Load Case 3: Gravity Loads and Lateral Loads

CD	1.60	(Wind/Seismic)
Mmax	185.28	ft-lbs
	2223.36	in-lbs
CL	0.99	
Cr	1.15	@ 16 O/C
Fbx'	1278.76	psi
fbx	294.00	psi
CSI (bending MWFRFS)	**0.23**	**OK**

Bearing on Stud Wall Plates

lb	1.50	in
Cb	1.00	(conservative)
Fc⊥'	405.00	psi
fc⊥	99.56	psi
CSI (bearing)	**0.25**	**OK**

Combined Stress

(re-evaluate compression values with CD = 1.6)

FcEx	1376.02	psi
FcE	1376.02	psi
Fc*	1360.00	psi
c	0.80	sawn lumber
FcE/Fc*	1.012	
1 + FcE/Fc*/2c	1.257	
Cp	0.695	
Fc'	945.22	psi

Deflection

E'	1,400,000	psi
ΔWIND (.42C&C)*	0.07	in
L/d**	**1317**	**OK**

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}'}\right) = \mathbf{0.26}\ \textbf{OK}$$

*IBC 2015 Sec. 1604.3

**IRC 2015 Sec. 301.7

Load Case: LCMAX

*LCMAX takes 100% of all loads for axial and bending.

Location:	Wall A
Specification:	**Use 2X6 DF Stud Grade @ 16" o/c**

Rev. 1.3.1 - 06/23/2015

Job#: **2015-035**

TRUSS UPLIFT CALCULATIONS

Out-to-out Span	42.875	ft	Load Combo:	.6D + .6W
Overhang Left	1.33	ft		
Overhang Right	1.33	ft		
Truss O/C Spacing	2	ft		
Roof Pitch	4	:12	18.43	Deg.

Dead Loads

BCDL	10	psf	SF (Slope Factor) =	1.05	
TCDL	7	psf	Adj. TCDL =	7.38	psf

Wind Loads (MWFRS)

Wind (0 to h/2)	31.81	psf
Wind (0 to h/2 @ overhang)	25.75	psf
Wind (overhang underside)	22.89	psf

Moment Summation at Truss Left Bearing:

	Force		Moment-Arm		Moment	
BCDL	-514.5	lbs	21.4	ft	-11029.6	ft-lbs
TCDL	-379.6	lbs	21.4	ft	-8138.3	ft-lbs
TCDL (right overhang)	-11.8	lbs	43.5	ft	-512.7	ft-lbs
TCDL (left overhang)	-11.8	lbs	-0.7	ft	7.8	ft-lbs
Wind (0 to h/2)	1636.6	lbs	21.4	ft	35085.1	ft-lbs
Wind (right overhang)	77.6	lbs	43.5	ft	3380.0	ft-lbs
Wind (left overhang)	77.6	lbs	-0.7	ft	-51.6	ft-lbs
Totals	874.2	lbs	150.1		18740.6	ft-lbs

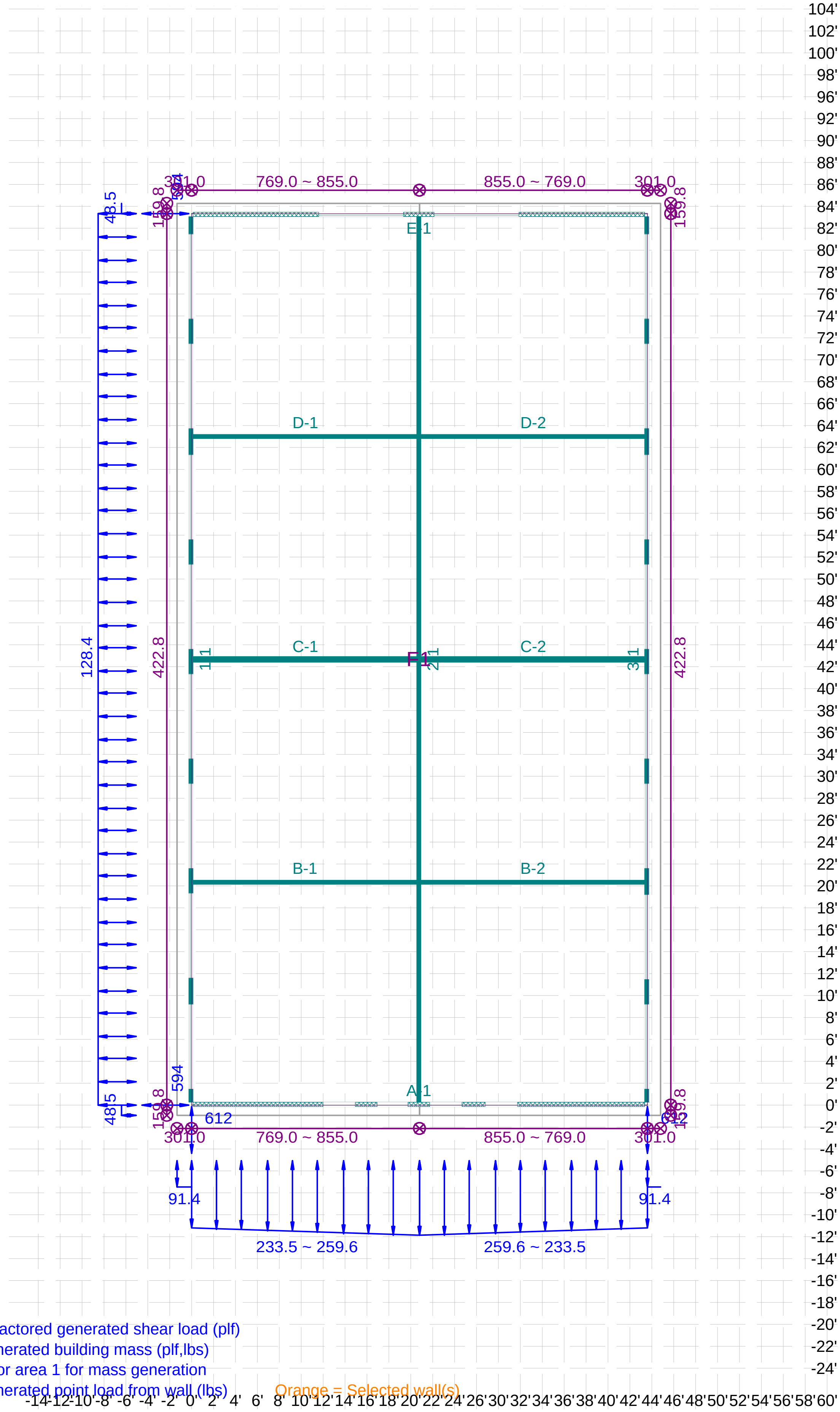
Uplift Right	**437.1**	**lbs**
Uplift Left	**437.1**	**lbs**

Use H1 Hurricane Ties for all other rafters and trusses.

H1 Allowable	585	lbs
H8 Allowable	745	lbs
H10A Allowable	1140	lbs
LGT2 Allowable	2050	lbs
LGT3 Allowable	3685	lbs

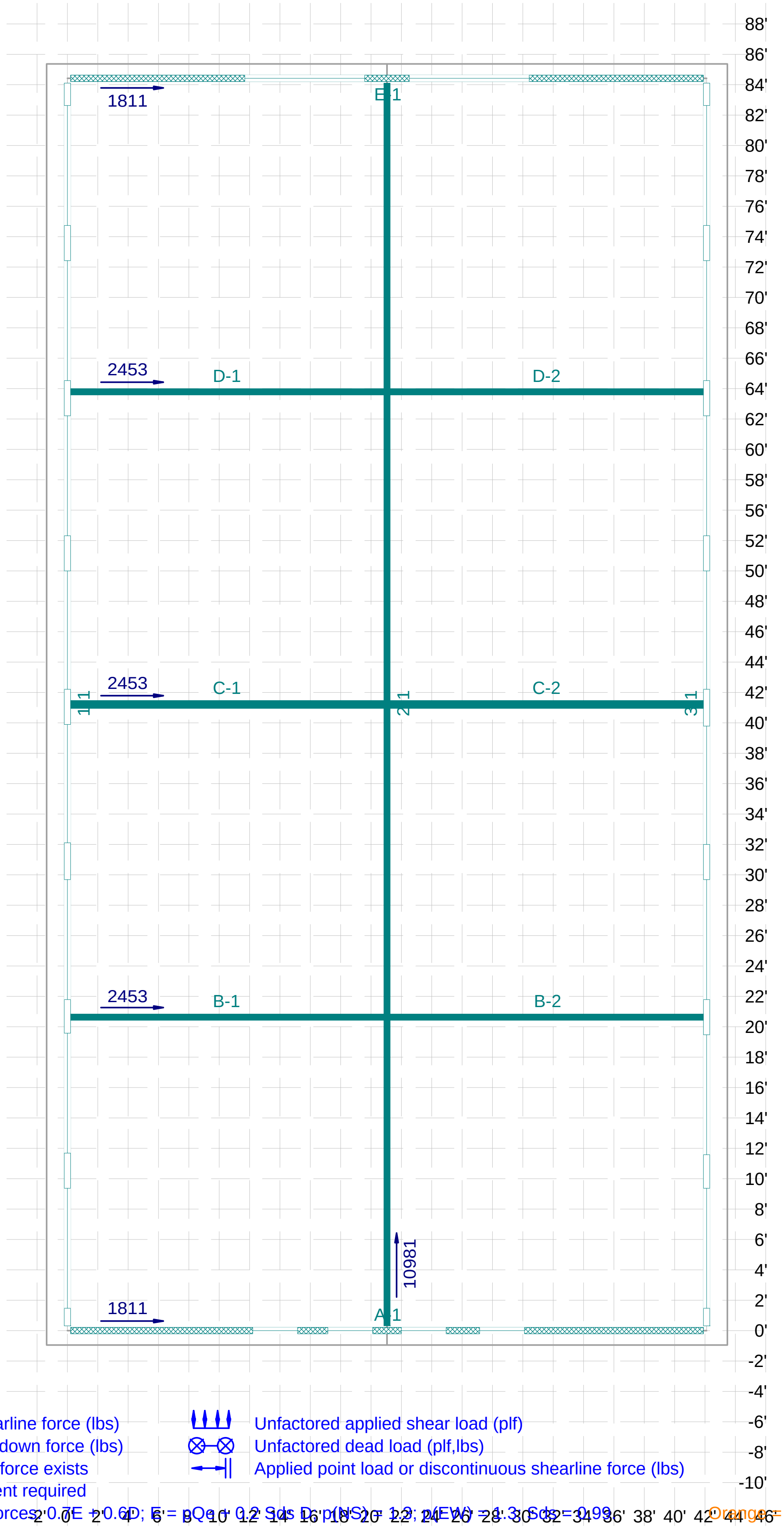
Note: Uplift allowable values are for DF/SP lumber and have been increased for wind or seismic (1.6).

Level 1 of 1



Unfactored generated shear load (plf)
Generated building mass (plf,lbs)
F1 - Floor area 1 for mass generation
Generated point load from wall (lbs)
Orange = Selected wall(s)

Level 1 of 1



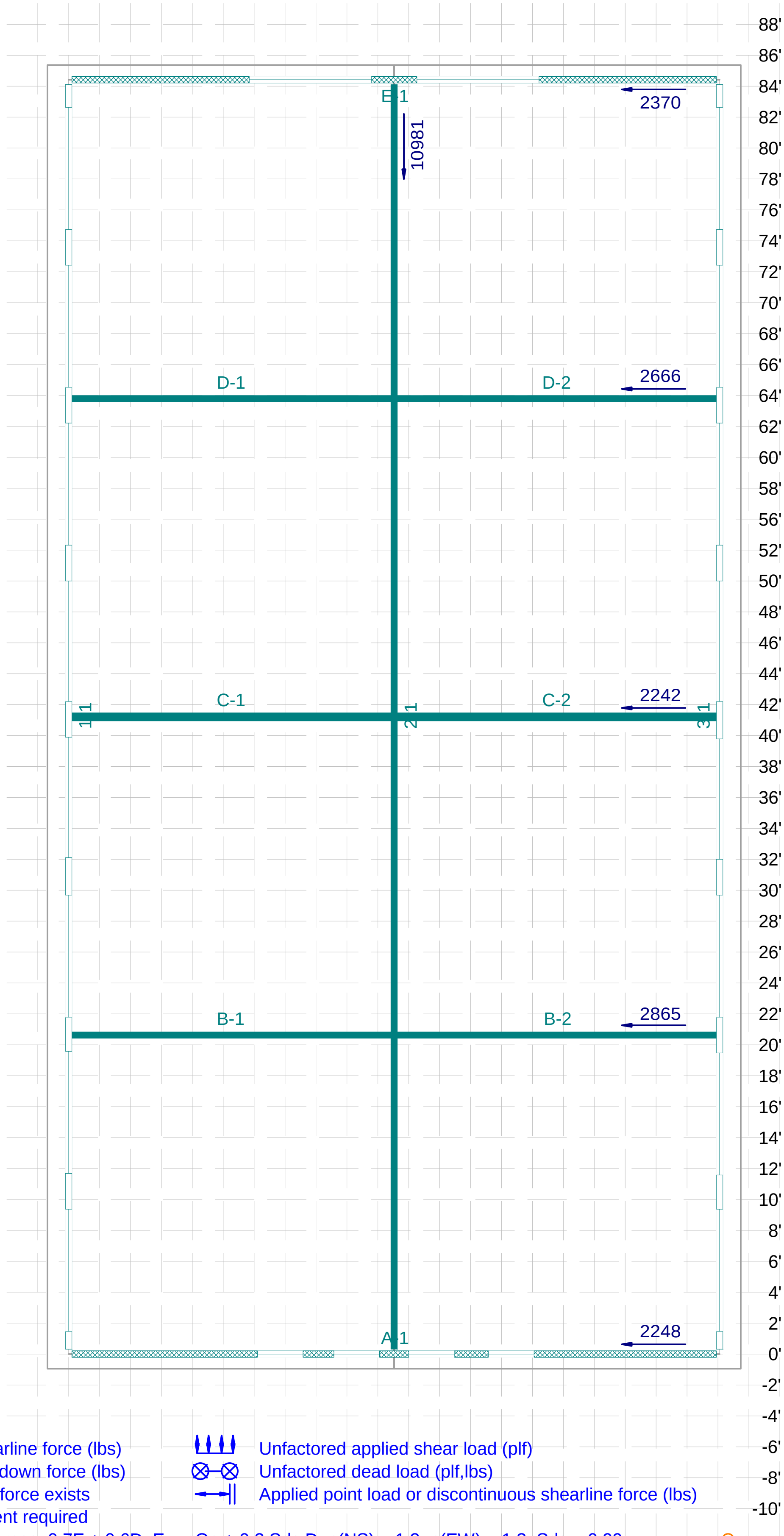
Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
Vertical element required

Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: Qe; Forces: 0.7E + 0.6D; E = pQe + 0.2 Sds D; p(NS) = 1.3; p(EW) = 1.3; Sds = 0.99

Orange = Selected wall(s)

Level 1 of 1



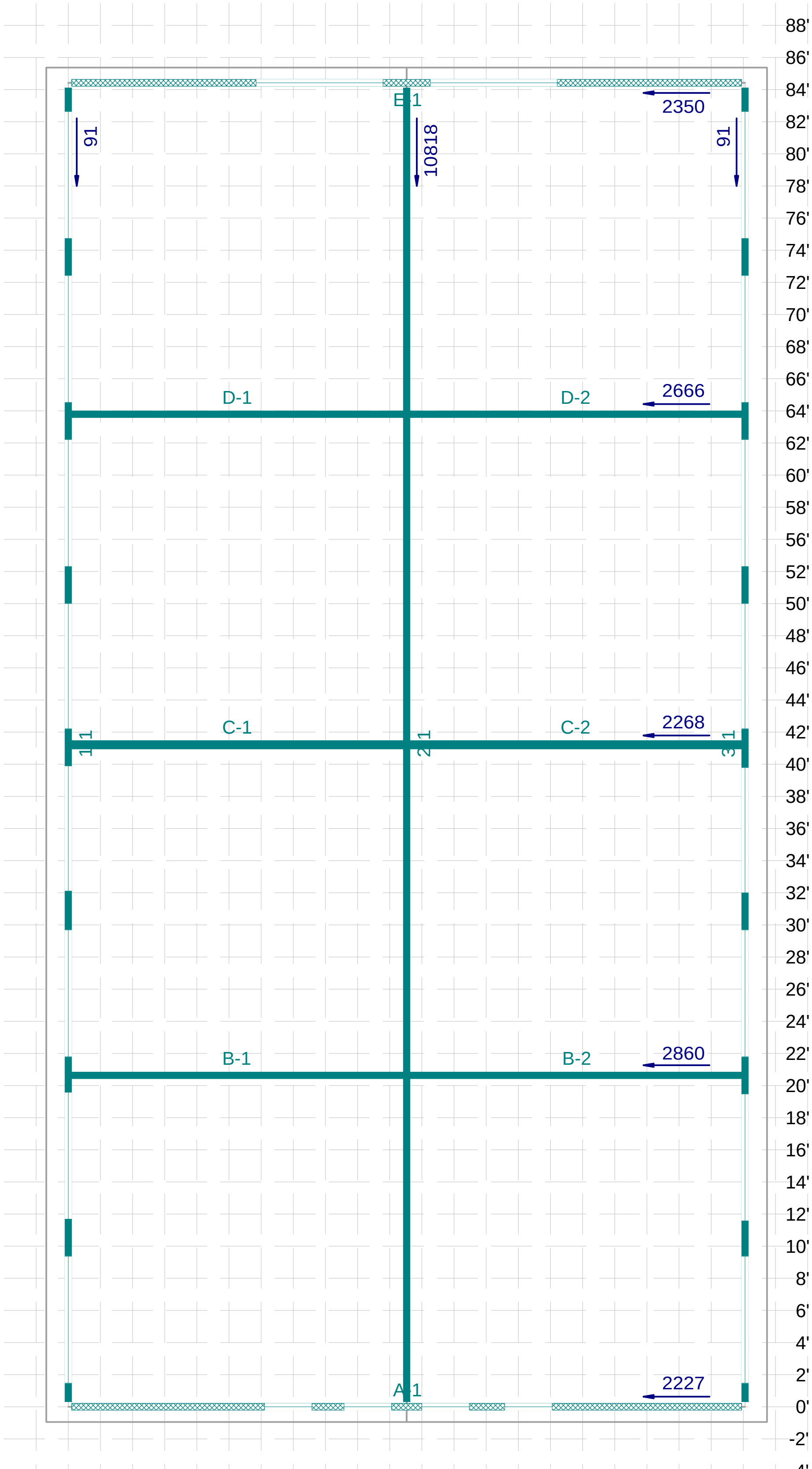
Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: Qe; Forces: 0.7E + 0.6D; E = ρQe + 0.2 Sds D; ρ(NS) = 1.3; ρ(EW) = 1.3; Sds = 0.99

Orange = Selected wall(s)

WoodWorks® Shearwalls	SOFTWARE FOR WOOD DESIGN

Level 1 of 1



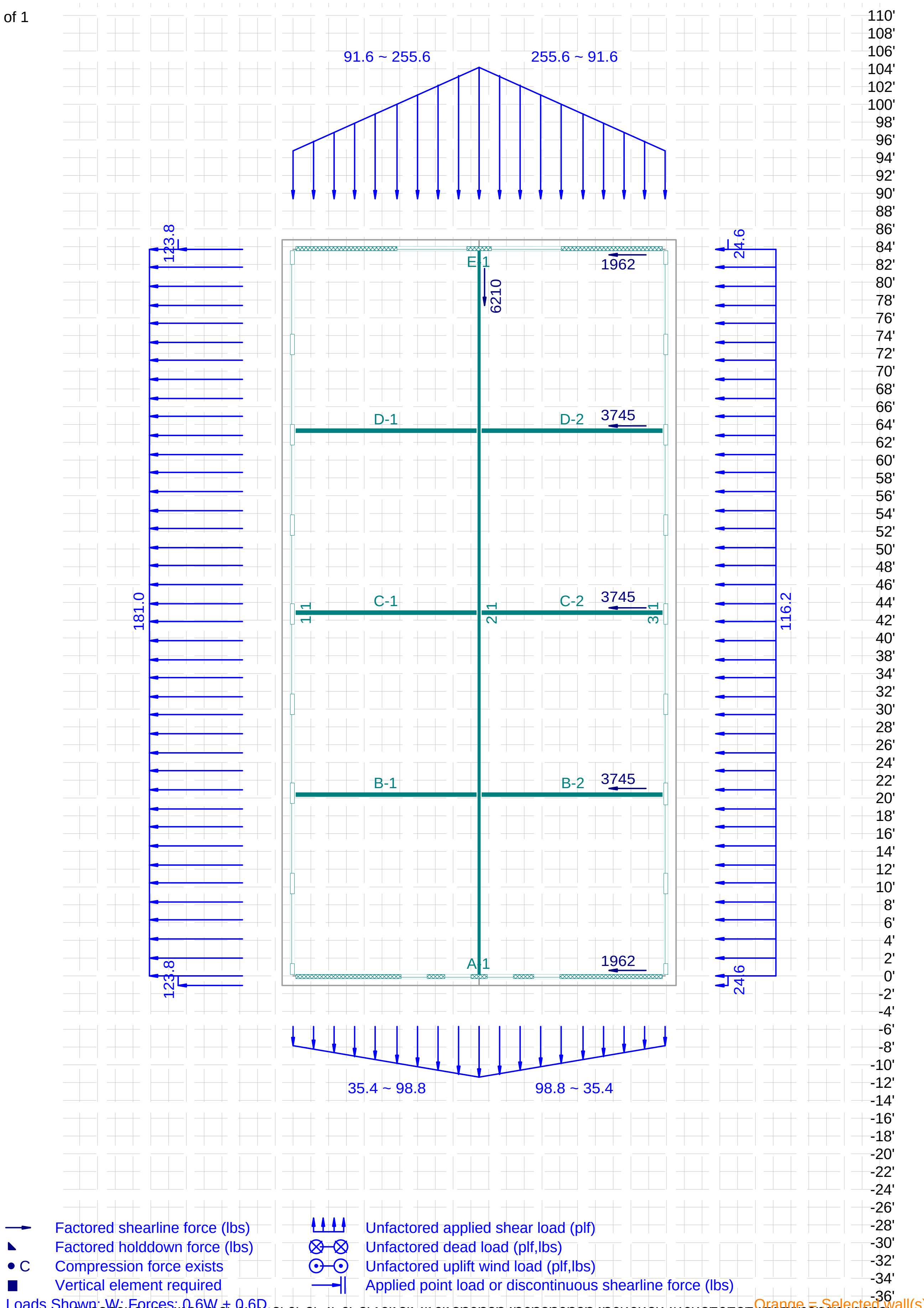
Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: Qe; Forces: 0.7E + 0.6D; E = ρQe + 0.2 Sds D; ρ(NS) = 1.3; ρ(EW) = 1.3; Sds = 0.99

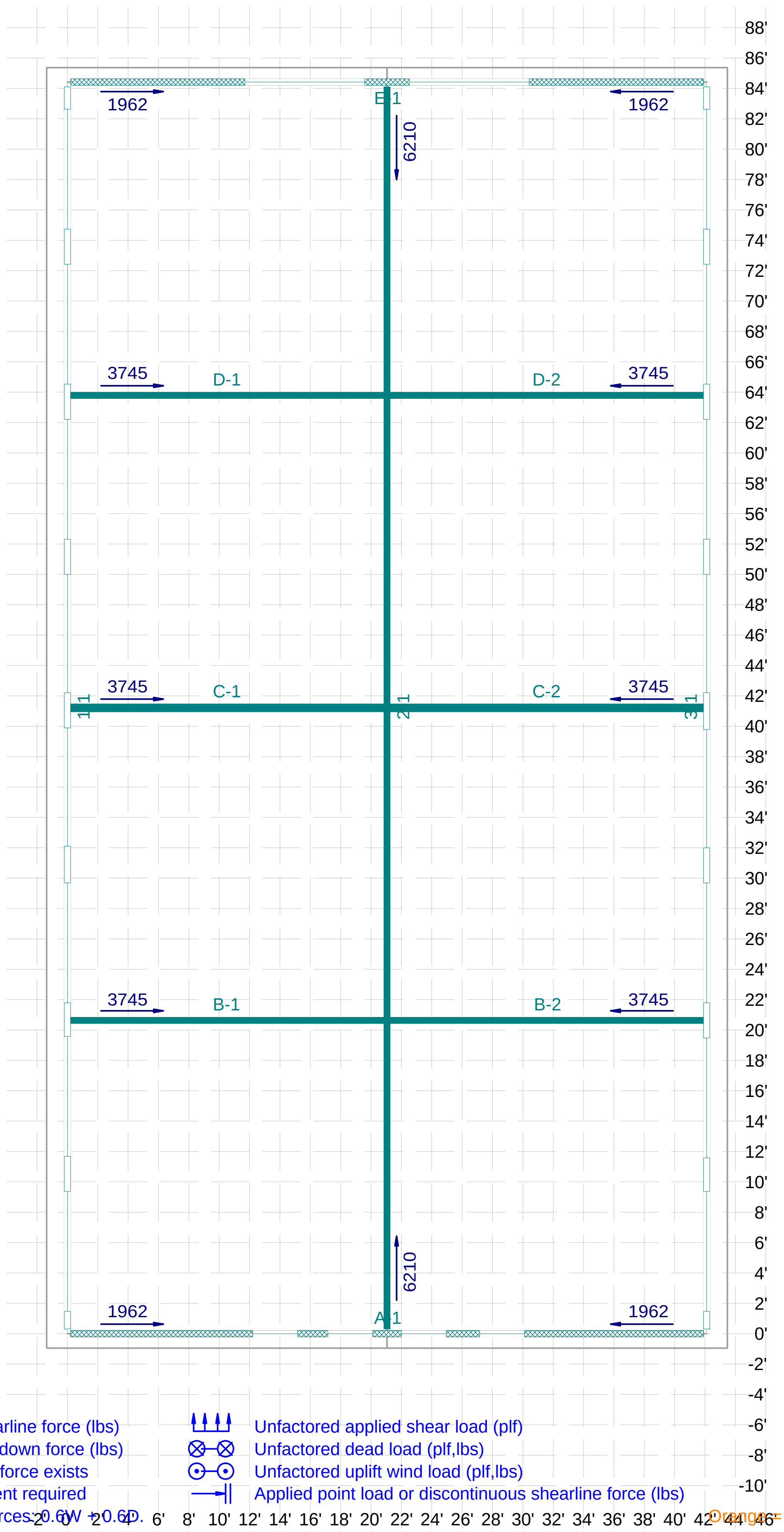
Orange = Selected wall(s)

WoodWorks® Shearwalls SOFTWARE FOR WOOD DESIGN

Level 1 of 1



Level 1 of 1



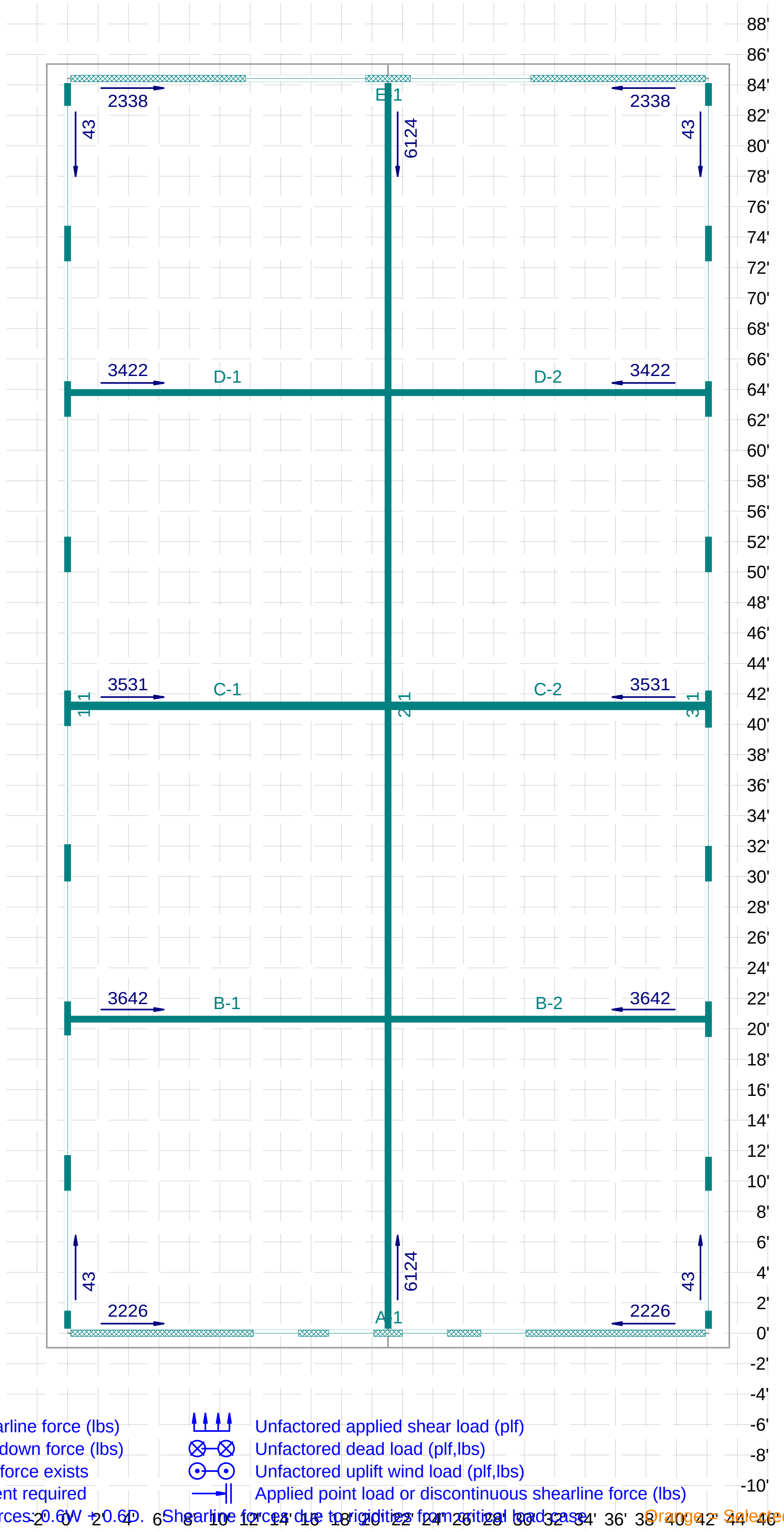
Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Unfactored uplift wind load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: W; Forces: 0.6W + 0.6D.

Orange = Selected wall(s)

Level 1 of 1

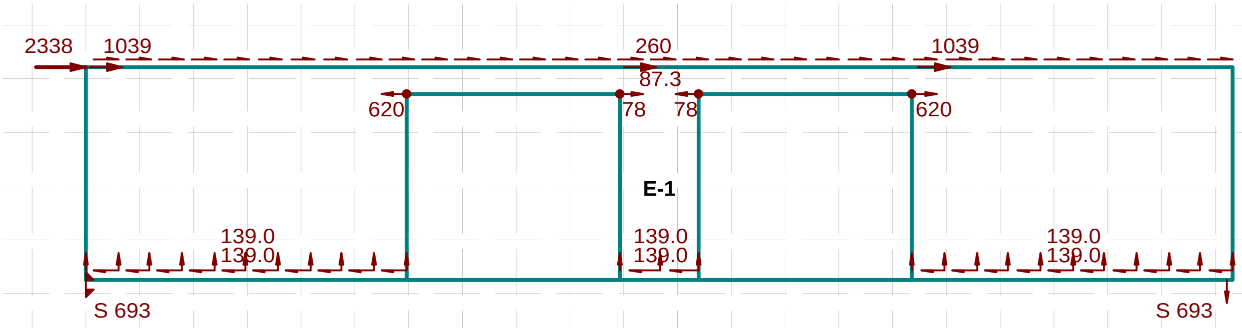
Loads Shown: W; Forces: 0.6W + 0.6D. Shearline forces due to rigidities from critical load case. Orange = Selected wall(s)



Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Unfactored uplift wind load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: W; Forces: 0.6W + 0.6D. Shearline forces due to rigidities from critical load case. Orange = Selected wall(s)

Elevation View
Shearline E, at Y = 84 ft, Level 1.
Rigid Diaphragm Wind Design.



Perforated, Co = 0.62

All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 226.8 plf
Interior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 226.8 plf
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall E-1 (cap. includes Co):
Design shear force: 86.6 plf
Combined capacity (added): 453.7 plf

Factored Forces

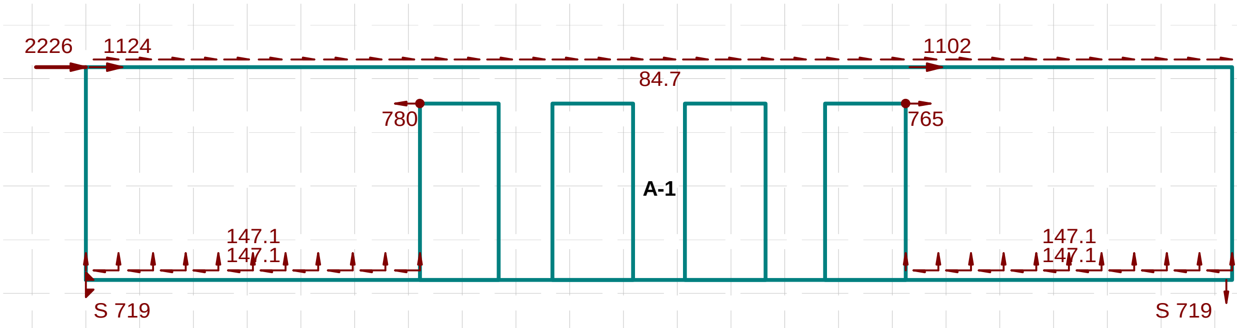
Vertical
Holddown force (lbs)
Compression force (lbs)
Anchorage force (t) (plf)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)
Combined: S - D + U (tens); S + D - U (comp)
S divided by perforation factor Co.

Horizontal
Vs - Shearline force (lbs)
Vs / diaphragm length / Co (plf)
V / full height sheathing / Co (plf)
Drag strut force (lbs)
Factors: S,U = 0.6
D = 0.6 (tens); 1.0 (comp)

Unfactored Loads
Dead
Wind uplift

West **East**

Elevation View
Shearline A, at Y = 0 ft, Level 1.
Rigid Diaphragm Wind Design.



Perforated, Co = 0.61

All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 222.6 plf
Interior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 222.6 plf
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall A-1 (cap. includes Co):
Design shear force: 89.9 plf
Combined capacity (added): 445.1 plf

Factored Forces

Vertical
Holddown force (lbs)
Compression force (lbs)
Anchorage force (t) (plf)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline force (lbs)
Vs / diaphragm length / Co (plf)
V / full height sheathing / Co (plf)
Drag strut force (lbs)
Factors: S,U = 0.6
D = 0.6 (tens); 1.0 (comp)

Combined: S - D + U (tens); S + D - U (comp)
S divided by perforation factor Co.

Unfactored Loads
Dead
Wind uplift

West **East**

SHEARWALL SUMMARY

SWL	Wind Flex.	Wind Rigid	Wind Max.	Wind Avg.	Description
1	1,962	2,338	**2,338**	**2,150**	PERFORATED - Co = 0.62
2	3,745	3,420	**3,745**	**3,583**	SEGMENTED
3	3,745	3,531	**3,745**	**3,638**	SEGMENTED
4	3,745	3,643	**3,745**	**3,694**	SEGMENTED
5	1,962	2,226	**2,226**	**2,094**	PERFORATED - Co = 0.61
B	6,210	6,210	**6,210**	**6,210**	SEGMENTED

SWL	Seismic Flex.	Seismic Rigid	Seismic Max.	Seismic Avg.	Description
1	1,811	2,370	**2,370**	**2,091**	PERFORATED - Co = 0.62
2	2,453	2,666	**2,666**	**2,560**	SEGMENTED
3	2,453	2,242	**2,453**	**2,348**	SEGMENTED
4	2,453	2,865	**2,865**	**2,659**	SEGMENTED
5	1,811	2,248	**2,248**	**2,030**	PERFORATED - Co = 0.61
B	10,981	10,981	**10,981**	**10,981**	SEGMENTED

Comments: Gable Trusses above SWL1, SWL2 capable of lateral load from shearwall (sheathed).
Interior sheathing of ceiling diaphragm distributes loads into interior shearwalls.

Design Method	Allowable Stress Design (ASD) ▼
Connection Type	Lateral loading ▼
Fastener Type	Bolt ▼
Loading Scenario	Single Shear - Concrete Main Member ▼

Main Member Type	Concrete ▼
Bolt Embedment Depth in Concrete	-- Other (in inches) -- ▼ 4
Main Member: Angle of Load to Grain	0
Side Member Type	Northern Species ▼
Side Member Thickness	1.5 in. ▼
Side Member: Angle of Load to Grain	0
Fastener Diameter	3/8 in. ▼
Load Duration Factor	C_D = 1.6 ▼
Wet Service Factor	C_M = 1.0 ▼
Temperature Factor	C_t = 1.0 ▼

Connection Yield Modes

Im	4500 lbs.
Is	878 lbs.
II	1483 lbs.
IIIm	1686 lbs.
IIIs	540 lbs.
IV	617 lbs.

Adjusted ASD Capacity	**540 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.

While every effort has been made to insure the accuracy of the information presented, and special effort has been made to assure that the information reflects the state-of-the-art, neither the American Wood Council nor its members assume any responsibility for any particular design prepared from this on-line Connection Calculator. Those using this on-line Connection Calculator assume all liability from its use.

The Connection Calculator was designed and created by Cameron Knudson, Michael Dodson and David Pollock at Washington State University. Support for development of the Connection Calculator was provided by American Wood Council.

Design Method	Allowable Stress Design (ASD) ▼
Connection Type	Lateral loading ▼
Fastener Type	Bolt ▼
Loading Scenario	Single Shear - Concrete Main Member ▼

Main Member Type	Concrete ▼
Bolt Embedment Depth in Concrete	-- Other (in inches) -- ▼ 4
Main Member: Angle of Load to Grain	0
Side Member Type	Northern Species ▼
Side Member Thickness	1.5 in. ▼
Side Member: Angle of Load to Grain	90
Fastener Diameter	3/8 in. ▼
Load Duration Factor	C_D = 1.6 ▼
Wet Service Factor	C_M = 1.0 ▼
Temperature Factor	C_t = 1.0 ▼

Connection Yield Modes

Im	3600 lbs.
Is	387 lbs.
II	1057 lbs.
IIIm	1152 lbs.
IIIs	303 lbs.
IV	398 lbs.

Adjusted ASD Capacity	**303 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.

The Connection Calculator was designed and created by Cameron Knudson, Michael Dodson and David Pollock at Washington State University. Support for development of the Connection Calculator was provided by American Wood Council.

SHEAR WALL CALCULATOR | **SWL1** | Vs = (seismic) 2370 lbs | Vw = (wind) 2338 lbs | **Job#: 2015-035**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL1	2,370	42.9	42.9	142.2	8.1	1,152	DTT2Z	THD501200H	5	(2) 2x6	1
PERF	Seismic Load Governs		Co =	0.62	ΣLi (ft) =	26.88	bs =	2.75	ft	DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **142** plf < v_{allow} = 163 plf ⟶ OK (seismic)

v_w = **140** plf < v_{allow} = 335 plf ⟶ OK (wind)

Edge Nail Spacing = 6 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 2.75 ft
Max. AR: h/bs = **2.95** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = 0.68

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x
AB DIA = 0.5 in
Zpara = 530 lbs
Zperp = 290 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 848 lbs
Zperp = 464 lbs

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf
SDS = 0.984 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 8.1 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 574 lbs

Out-of-Plane Wind (MWFRS)

Ww = 28.95 psf
Ltrib = 4.05 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 3,016 lbs

Wind Load Governs:
V_{perp} = 3,016 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	3,016	6.5	**4.1**
Para. Load	2,370	2.8	**9.6**

La = 26.9 ft La = available wall length for anchor bolts

Use 1/2" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 48" o/c spacing all of Wall 1.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 42.9 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 55.3 plf
Spacing = **10.9** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 199.1 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.128 in (Simpson Holdown)
en = 0.0049 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	42.875	0.09	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.09**	in

Cd = 4
ASCE 7-10 (Table 12.12-1) Δ = 0.35 in
Δlimit = 1.944 in ⟶ OK

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: 2015-035

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
$F_{c\perp}$	405	psi
$C_{t\_c\perp}$	1.00	
$C_{M\_c\perp}$	1.00	
C_b	1.00	(1.125)
$F_{c\perp}'$	405.00	psi
A_b	16.50	in^2
P_c	1264	lbs
$f_{c\perp}$	77	psi
CSI (bearing)	**0.19**	→ **OK**

Chord in Tension

(DF No. 2)

F_t	575	psi
C_{M_t}	1.00	
C_{t_t}	1.00	
C_{i_t}	1.00	
C_D	1.60	(seismic)
C_{F_t}	1.30	
F_t'	1196	psi
A_n	16.50	in^2
f_t	70	psi
CSI (tension)	**0.06**	→ **OK**

Chord in Compression

(DF No. 2)

F_c	1350	psi
C_{M_c}	1.00	
C_{t_c}	1.00	
C_{i_c}	1.00	
C_D	1.60	(seismic)
C_{F_c}	1.10	
$(l_e/d)_x$	16.85	
E'_{min}	580,000	psi
F_{cE}	1678	psi
F_c^*	2376	psi
c	0.80	sawn lumber
F_{cE}/F_c^*	0.706	
$1 + F_{cE}/F_c^*/2c$	1.066	
C_p	0.562	
F_c'	1335	psi
f_c	77	psi
CSI (compression)	**0.06**	→ **OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	W_{DL}	W_{LL}	W_{SL}/W_{LrL}
Wall Loads	84	0	45

(lbs)	P_{DL}	P_{LL}	P_{SL}/P_{LrL}	P_W (+/-)	P_S (+/-)
Point loads	0	0	0	0	0

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	1,249	plf
6a.) D + .75L + .75W + 75(Lr or S) =	1,009	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,264	plf (governs)
6b.) D + .75L + .75E + 75S =	1,021	plf

P_W =	1,137	lbs
P_S =	1,152	lbs
* SWL Chord Tension =	**1,152**	lbs
SWL Chord Comp. =	**1,264**	lbs
Stud Spacing =	16	in
Chord Studs =	(2) 2x6	
Chord Depth (dx) =	5.5	in
l_b =	3.00	in
E_{min} =	580,000	psi
C_{M_e} =	1.00	
C_{t_e} =	1.00	
C_{t_e} =	1.00	

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs (NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)
Z' =	226	lbs
Unit Shear =	142.2	plf
Spacing =	**19.0**	in

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL2**

Vs = 2666 lbs (seismic)

Vw = 3745 lbs (wind)

Job#: 2015-035

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL2	3,745	42.0	42.0	89.2	8.1	722	DTT2Z	THD501200H	5	(1) 2x4	1
SEGMENT	Wind Load Governs									DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **63** plf < v_{allow} = 240 plf → OK (seismic)

v_w = **89** plf < v_{allow} = 335 plf → OK (wind)

Edge Nail Spacing = 6 in

Sheathing both sides = NO

Sht. Panel Thickness = 7/16 in

Fastener Type = 8d

Min. Panel Length: bs = 42 ft

Max. AR: h/bs = **0.19** → OK

Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x

AB DIA = 0.5 in

Zpara = 530 lbs

Zperp = 290 lbs

Applying adjustment factors:

CD = 1.6 (wind or seismic)

Zpara = 848 lbs **(540 lbs)**

Zperp = 464 lbs **(303 lbs)**

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf

SDS = 0.984 g

Ie = 1.0

ka = 1.0 (concrete is rigid)

Wall Hgt. = 8.1 ft

ρ = 1.0 (out-of-plane)

Vsperp is given as the seismic force of half the dead weight of the wall.

V_{sperp} = 562 lbs

Out-of-Plane Wind (MWFRS)

Ww = 0 psf

Ltrib = 4.05 ft

Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.

V_{wperp} = 0 lbs

Seismic Load Governs:

V_{perp} = 562 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	562	1.2	**34.7**
Para. Load	3,745	4.4	**9.5**

(6.1) @ 3/8"

La = 42.0 ft La = available wall length for anchor bolts

Use 3/8" x 3-1/2" Powers SPIKE Anchor (Mushroom Head Carbon Steel), 1-3/4" min. embedment @ 54" o/c spacing all of Wall 2.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 42.0 ft (available collector length)

Fallow = 600 lbs (F1 direction)

Unit Shear = 89.2 plf

Spacing = **6.7** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 88.9 plf

E = 1,600,000 psi

A = 5.25 in^2

Gt = 83,500 plf (Table C4.2.2A)

da = 0.105 in (Simpson Holdown)

en = 0.0004 in (Table C4.2.2D)

nail spacing = 6 in

Sht. both sides = NO

Panel #	b (ft)	Δs	
1	42	0.03	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.03**	in

Cd = 4

ASCE 7-10 (Table 12.12-1)

Δ = 0.13 in

Δlimit = 1.944 in → OK

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: 2015-035

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
Fc⊥	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.250)
Fc⊥'	405.00	psi
Ab	5.25	in^2
Pc	722	lbs
fc⊥	138	psi
CSI (bearing)	**0.34**	→ **OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(wind)
CF_t	1.50	
Ft'	1380	psi
An	5.25	in^2
ft	138	psi
CSI (tension)	**0.10**	→ **OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(wind)
CF_c	1.15	
(le/d)x	26.49	
E'min	580,000	psi
FcE	680	psi
Fc*	2484	psi
c	0.80	sawn lumber
FcE/Fc*	0.274	
1 + FcE/Fc*/2c	0.796	
Cp	0.256	
Fc'	636	psi
fc	138	psi
CSI (compression)	**0.22**	→ **OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	0	0	0

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	0	0	0	0	0

Pw = 722 lbs
Ps = 514 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	722	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	542	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	514	plf
6b.) D + .75L + .75E + 75S =	386	plf

* SWL Chord Tension =	**722**	lbs
SWL Chord Comp. =	**722**	lbs
Stud Spacing =	16	in
Chord Studs =	(1) 2x4	
Chord Depth (dx) =	3.5	in
lb =	1.50	in

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs (NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)
Z' =	226	lbs
Unit Shear =	89.2	plf
Spacing =	**30.4**	in

Emin =	580,000	psi
CM_e =	1.00	
Ct_e =	1.00	
Ct_e =	1.00	

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL3** Vs = 2453 lbs (seismic) Vw = 3745 lbs (wind) **Job#: 2015-035**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL3	3,745	42.0	42.0	89.2	8.1	722	DTT2Z	THD501200H	5	(1) 2x4	1
SEGMENT	Wind Load Governs									DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **58** plf < v_{allow} = 240 plf → OK (seismic)

v_w = **89** plf < v_{allow} = 335 plf → OK (wind)

Edge Nail Spacing = 6 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 42 ft
Max. AR: h/bs = **0.19** → OK
Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x
AB DIA = 0.5 in
Zpara = 530 lbs
Zperp = 290 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 848 lbs **(540 lbs)**
Zperp = 464 lbs **(303 lbs)**

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf
SDS = 0.984 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 8.1 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 562 lbs

Out-of-Plane Wind (MWFRS)

Ww = 0 psf
Ltrib = 4.05 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 0 lbs

Seismic Load Governs:
V_{perp} = 562 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	562	1.2	**34.7**
Para. Load	3,745	4.4	**9.5**

(6.1) @ 3/8"

La = 42.0 ft La = available wall length for anchor bolts

Use 3/8" x 3-1/2" Powers SPIKE Anchor (Mushroom Head Carbon Steel), 1-3/4" min. embedment @ 54" o/c spacing all of Wall 3.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 42.0 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 89.2 plf
Spacing = **6.7** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 81.8 plf
E = 1,600,000 psi
A = 5.25 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.105 in (Simpson Holdown)
en = 0.0003 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	42	0.03	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.03**	in

Cd = 4
Δ = 0.12 in
ASCE 7-10 (Table 12.12-1) Δlimit = 1.944 in → OK

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: **2015-035**

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
Fc⊥	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.250)
Fc⊥'	405.00	psi
Ab	5.25	in^2
Pc	722	lbs
fc⊥	138	psi
CSI (bearing)	**0.34**	→ **OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(wind)
CF_t	1.50	
Ft'	1380	psi
An	5.25	in^2
ft	138	psi
CSI (tension)	**0.10**	→ **OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(wind)
CF_c	1.15	
(le/d)x	26.49	
E'min	580,000	psi
FcE	680	psi
Fc*	2484	psi
c	0.80	sawn lumber
FcE/Fc*	0.274	
1 + FcE/Fc*/2c	0.796	
Cp	0.256	
Fc'	636	psi
fc	138	psi
CSI (compression)	**0.22**	→ **OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	0	0	0

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	0	0	0	0	0

Pw = 722 lbs

Ps = 473 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	722	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	542	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	473	plf
6b.) D + .75L + .75E + 75S =	355	plf

* SWL Chord Tension =	**722**	lbs
SWL Chord Comp. =	**722**	lbs
Stud Spacing =	16	in
Chord Studs =	(1) 2x4	
Chord Depth (dx) =	3.5	in
lb =	1.50	in

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs	(NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)	
Z' =	226	lbs	
Unit Shear =	89.2	plf	
Spacing =	**30.4**	in	

Emin =	580,000	psi
CM_e =	1.00	
Ct_e =	1.00	
Ct_e =	1.00	

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL4** Vs = 2865 lbs (seismic) Vw = 3745 lbs (wind) **Job#: 2015-035**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL4	3,745	42.0	42.0	89.2	8.1	722	DTT2Z	THD501200H	5	(1) 2x4	1
SEGMENT	Wind Load Governs									DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **68** plf < v_{allow} = 240 plf → OK (seismic)

v_w = **89** plf < v_{allow} = 335 plf → OK (wind)

Edge Nail Spacing = 6 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 42 ft
Max. AR: h/bs = **0.19** → OK
Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x
AB DIA = 0.5 in
Zpara = 530 lbs
Zperp = 290 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 848 lbs **(540 lbs)**
Zperp = 464 lbs **(303 lbs)**

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf
SDS = 0.984 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 8.1 ft
ρ = 1.0 (out-of-plane)

Vsperp is given as the seismic force of half the dead weight of the wall.

V_{sperp} = 562 lbs

Out-of-Plane Wind (MWFRS)

Ww = 0 psf
Ltrib = 4.05 ft

Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.

V_{wperp} = 0 lbs

Seismic Load Governs:

V_{perp} = 562 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	562	1.2	**34.7**
Para. Load	3,745	4.4	**9.5**

(6.1) @ 3/8"

La = 42.0 ft La = available wall length for anchor bolts

Use 3/8" x 3-1/2" Powers SPIKE Anchor (Mushroom Head Carbon Steel), 1-3/4" min. embedment @ 54" o/c spacing all of Wall 4.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 42.0 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 89.2 plf
Spacing = **6.7** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 95.5 plf
E = 1,600,000 psi
A = 5.25 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.105 in (Simpson Holdown)
en = 0.0005 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	42	0.03	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.03**	in

Cd = 4
ASCE 7-10 Δ = 0.14 in
(Table 12.12-1) Δlimit = 1.944 in → OK

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: 2015-035

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
Fc⊥	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.250)
Fc⊥'	405.00	psi
Ab	5.25	in^2
Pc	722	lbs
fc⊥	138	psi
CSI (bearing)	**0.34**	→ **OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(wind)
CF_t	1.50	
Ft'	1380	psi
An	5.25	in^2
ft	138	psi
CSI (tension)	**0.10**	→ **OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(wind)
CF_c	1.15	
(le/d)x	26.49	
E'min	580,000	psi
FcE	680	psi
Fc*	2484	psi
c	0.80	sawn lumber
FcE/Fc*	0.274	
1 + FcE/Fc*/2c	0.796	
Cp	0.256	
Fc'	636	psi
fc	138	psi
CSI (compression)	**0.22**	→ **OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	0	0	0

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	0	0	0	0	0

Pw =	722	lbs
Ps =	553	lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	722	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	542	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	553	plf
6b.) D + .75L + .75E + 75S =	414	plf

* SWL Chord Tension =	**722**	lbs
SWL Chord Comp. =	**722**	lbs
Stud Spacing =	16	in
Chord Studs =	(1) 2x4	
Chord Depth (dx) =	3.5	in
lb =	1.50	in
Emin =	580,000	psi
CM_e =	1.00	
Ct_e =	1.00	
Ct_e =	1.00	

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs (NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)
Z' =	226	lbs
Unit Shear =	89.2	plf
Spacing =	**30.4**	in

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL5** Vs = 2248 lbs (seismic) Vw = 2226 lbs (wind) **Job#: 2015-035**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL5	2,248	42.9	42.9	156.8	8.1	1,270	DTT2Z	THD501200H	5	(2) 2x6	1
PERF	Seismic Load Governs		Co =	0.61	ΣLi (ft) =	23.50	bs =	11.75	ft	DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **157** plf < v_{allow} = 240 plf ⟶ **OK** (seismic)

v_w = **155** plf < v_{allow} = 335 plf ⟶ **OK** (wind)

Edge Nail Spacing = 6 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 11.75 ft
Max. AR: h/bs = **0.69** ⟶ **OK**
Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x
AB DIA = 0.5 in
Zpara = 530 lbs
Zperp = 290 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 848 lbs
Zperp = 464 lbs

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf
SDS = 0.984 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 8.1 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 574 lbs

Out-of-Plane Wind (MWFRS)

Ww = 28.95 psf
Ltrib = 4.05 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 3,016 lbs

Wind Load Governs:
V_{perp} = 3,016 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	3,016	6.5	**4.1**
Para. Load	2,248	2.7	**10.1**

La = 26.9 ft La = available wall length for anchor bolts

Use 1/2" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 48" o/c spacing all of Wall 1.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 42.9 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 52.4 plf
Spacing = **11.4** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 219.5 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.128 in (Simpson Holdown)
en = 0.0066 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	42.875	0.11	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.11**	in

Cd = 4
ASCE 7-10 (Table 12.12-1) Δ = 0.42 in
Δlimit = 1.944 in ⟶ **OK**

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: 2015-035

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
$Fc_{\perp}$	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.125)
$Fc_{\perp}'$	405.00	psi
Ab	16.50	in^2
Pc	1382	lbs
$fc_{\perp}$	84	psi
CSI (bearing)	**0.21**	→ **OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(seismic)
CF_t	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	77	psi
CSI (tension)	**0.06**	→ **OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(seismic)
CF_c	1.10	
(le/d)x	16.85	
E'min	580,000	psi
FcE	1678	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.706	
1 + FcE/Fc*/2c	1.066	
Cp	0.562	
Fc'	1335	psi
fc	84	psi
CSI (compression)	**0.06**	→ **OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	84	0	45

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	0	0	0	0	0

Pw =	1,258	lbs
Ps =	1,270	lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	1,370	plf
6a.) D + .75L + .75W + 75(Lr or S) =	1,100	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,382	plf (governs)
6b.) D + .75L + .75E + 75S =	1,110	plf

* SWL Chord Tension =	**1,270**	lbs
SWL Chord Comp. =	**1,382**	lbs
Stud Spacing =	16	in
Chord Studs =	(2) 2x6	
Chord Depth (dx) =	5.5	in
lb =	3.00	in

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs (NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)
Z' =	226	lbs
Unit Shear =	156.8	plf
Spacing =	**17.3**	in

Emin =	580,000	psi
CM_e =	1.00	
Ct_e =	1.00	
Ct_e =	1.00	

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWLB** Vs = 10981 lbs (seismic) Vw = 6210 lbs (wind) **Job#: 2015-035**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWLB	10,981	82.8	82.8	132.7	8.1	1,075	DTT2Z	THD501200H	5	(2) 2x4	1
SEGMENT	Seismic Load Governs									DF No. 2	

Shearwall Sheathing Specification:

Nominal unit shear capacities from SDPWS Table 4.3A (Wood Frame Shear Walls)

v_s = **133** plf < v_{allow} = 240 plf ⟶ OK (seismic)

v_w = **75** plf < v_{allow} = 335 plf ⟶ OK (wind)

Edge Nail Spacing = 6 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 82.75 ft
Max. AR: h/bs = **0.10** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field, blocking required.

Anchor Bolt Spacing

Since we cannot control species of pressure treated sill plate assume weakest species from NDS 2012 Table 11E for anchor bolts (Northern Species G = 0.35):

Sill Plate: (1)-2x
AB DIA = 0.5 in
Zpara = 530 lbs
Zperp = 290 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 848 lbs
Zperp = 464 lbs

Out-of-Plane Seismic

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

ASCE 7-10 Sec. 12.11.2

WDL = 12 psf
SDS = 0.984 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 8.1 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 1,108 lbs

Out-of-Plane Wind (MWFRS)

Ww = 0 psf
Ltrib = 4.05 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 0 lbs

Seismic Load Governs:
V_{perp} = 1,108 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	1,108	2.4	**34.7**
Para. Load	10,981	12.9	**6.4**

La = 82.8 ft La = available wall length for anchor bolts

Use 1/2" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 60" o/c spacing all of Wall B.

A35 Framing Angle Spacing

Provide full depth blocking with A35 clips to top plt. per plan.

Lac = 82.8 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 132.7 plf
Spacing = **4.5** ft

7/16" OSB sheathing on ceiling provides adequate connection, A35 angle clips not required.

Deflection (based on strength-level seismic forces)

v_u = 185.8 plf
E = 1,600,000 psi
A = 10.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.128 in (Simpson Holdown)
en = 0.0040 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	82.75	0.06	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.06**	in

ASCE 7-10 (Table 12.12-1)

Cd = 4
Δ = 0.22 in
Δlimit = 1.944 in ⟶ OK

General Notes:

1. For unblocked shearwalls w/ studs @ 16" o/c capacity is reduced by 0.6.
2. All stemwall foundations walls with HDU8 or greater holdown (anchor bolt ≥ 7/8" DIA) shall be 8" min. thickness.
3. Uplift on holdowns calculated with dead load counter action neglected (conservative).
4. Where the required nominal unit shear capacity on either side of a shear wall exceeds 700 plf in SDC D framing members at adjacent panel edges shall be 3X or double 2X.
6. All holdowns over TJI floor, use CNW coupler nut and threaded rod for extension. Solid squash blocks beneath all shearwall chords equal to chord cross section.

Job#: 2015-035

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
Fc⊥	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.125)
Fc⊥'	405.00	psi
Ab	10.50	in^2
Pc	1753	lbs
fc⊥	167	psi
CSI (bearing)	**0.41** →	**OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(seismic)
CF_t	1.50	
Ft'	1380	psi
An	10.50	in^2
ft	102	psi
CSI (tension)	**0.07** →	**OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(seismic)
CF_c	1.15	
(le/d)x	26.49	
E'min	580,000	psi
FcE	680	psi
Fc*	2484	psi
c	0.80	sawn lumber
FcE/Fc*	0.274	
1 + FcE/Fc*/2c	0.796	
Cp	0.256	
Fc'	636	psi
fc	167	psi
CSI (compression)	**0.26** →	**OK**

Shearwall Gravity Loads

(Point loads are assumed to bear directly above SWL chord)

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	382	0	438

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	0	0	0	0	0

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	1,117	plf
6a.) D + .75L + .75W + 75(Lr or S) =	1,403	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,584	plf
6b.) D + .75L + .75E + 75S =	1,753	plf (governs)

Pw =	608	lbs
Ps =	1,075	lbs
* SWL Chord Tension =	**1,075**	lbs
SWL Chord Comp. =	**1,753**	lbs
Stud Spacing =	16	in
Chord Studs =	(2) 2x4	
Chord Depth (dx) =	3.5	in
lb =	3.00	in

Bottom Plate (Sole Plt.) Attachment to Floor

This section is only applicable when shearwall is framed on top of a wood joist or TJI floor.

Z =	141	lbs (NDS 2012 Table 11Q for 16d nail, DF G = 0.5)
CD =	1.6	(wind or seismic)
Z' =	226	lbs
Unit Shear =	132.7	plf
Spacing =	**20.4**	in

Emin =	580,000	psi
CM_e =	1.00	
Ct_e =	1.00	
Ct_e =	1.00	

Slab-on-Grade Foundation, N/A

Sill Plate at Foundation

Use (1)-2x HF No. 2 pressure treated plate at foundation.

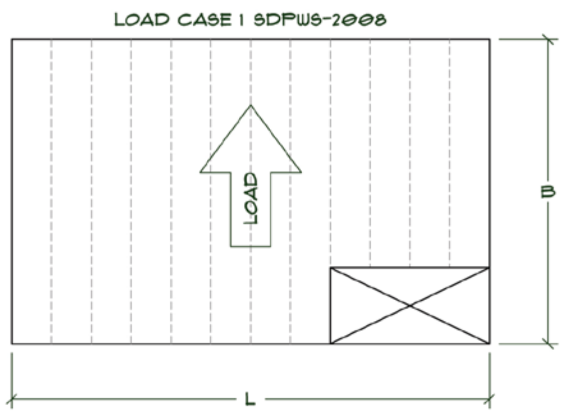
*Only applicable at first story shearwalls.

Roof Diaphragm and Sheathing Calculations

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

1.) Transverse Load Case:

W_D =	178 plf	L =	21 ft
Mmax =	9,812 ft-lbs	b =	43 ft
T=C=M/b=	**228 lbs**		
V =	1869 lbs	Load Case 1 SDPWS-2008	
v = V/b =	**43 plf**		

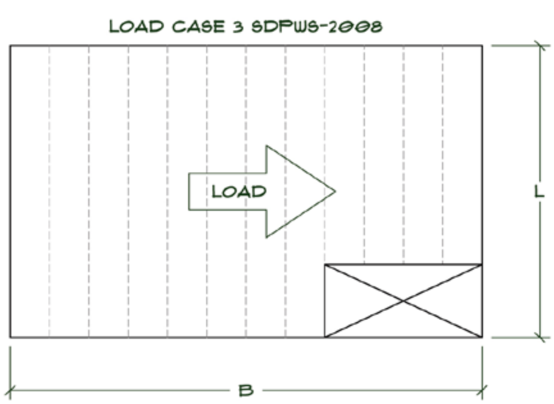


L/B Ratio = 0.49 → **OK**

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

2.) Longitudinal Load Case:

W_D =	213 plf	L =	21.5 ft
Mmax =	12,307 ft-lbs	b =	83 ft
T=C=M/b=	**148 lbs**		
V =	2290 lbs	Load Case 3 SDPWS-2008	
v = V/b =	**28 plf**		



L/B Ratio = 0.26 → **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:	SPF	
(Unblocked Diaphragm)			SGAF:	0.92	

Load Case 1: (transverse)

$v =$ 43 plf < $v_W =$ 297 plf ⟶ **OK**

Load Case 3: (longitudinal)

$v =$ 28 plf < $v_W =$ 219 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 = **228** 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' = 1.0 nails

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

Use (10) - 16d nails on each side of splice joint in wall top chords. Position splice joint over studs.

Where top chord is discontinous, apply an MSTC40 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	C
Basic Wind Speed (ultimate)	135.00 MPH

P_{3OH} = 110.21 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{3OH_ASD} = 66.126 psf

Roof Sheathing Nailing

	Edges (in.)	Field (in.)
Interior (Zone 1)	6	12
Perimeter (Zone 2)	6	6
Gable Endwall & Overhangs	4	4

*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 = 7.4 psf + 38.5 psf = 45.9 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 = 66.1 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	>	46.3 psf [2]	→	**OK**
L/180	→	68 psf	>	46.3 psf [2]	→	**OK**
Bending	→	128 psf	>	66.1 psf	→	**OK**
Shear	→	213 psf	>	66.1 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 @ 4" o/c edges, 4" 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:	7/16	in.
Nails:	8d	
Stud Spacing:	16	in. o/c

APA Rating: Grade 24/16

Sheathing Type: OSB

Terrain Exp. Category: C

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 = 47.18 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{5_ASD} = 28.308 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} = 28.3 psf < P_{allow} = 37.5 psf ⟶ **OK**

Sheathing Perpendicular to Studs

P_{5_ASD} = 28.3 psf < P_{allow} = 190.6 psf ⟶ **OK**

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

Sheathing Parallel to Studs						
L/360	⟶	26 psf	>	19.8 psf [2]	⟶	**OK**
Bending	⟶	86 psf	>	28.3 psf	⟶	**OK**
Shear	⟶	331 psf	>	28.3 psf	⟶	**OK**
Sheathing Perpendicular to Studs						
L/360	⟶	128 psf	>	19.8 psf [2]	⟶	**OK**
Bending	⟶	288 psf	>	28.3 psf	⟶	**OK**
Shear	⟶	331 psf	>	28.3 psf	⟶	**OK**

Sheath walls with 7/16 APA rated OSB (Grade 24/16) w/ 8d nails @ 6" o/c edges, 12" o/c field.

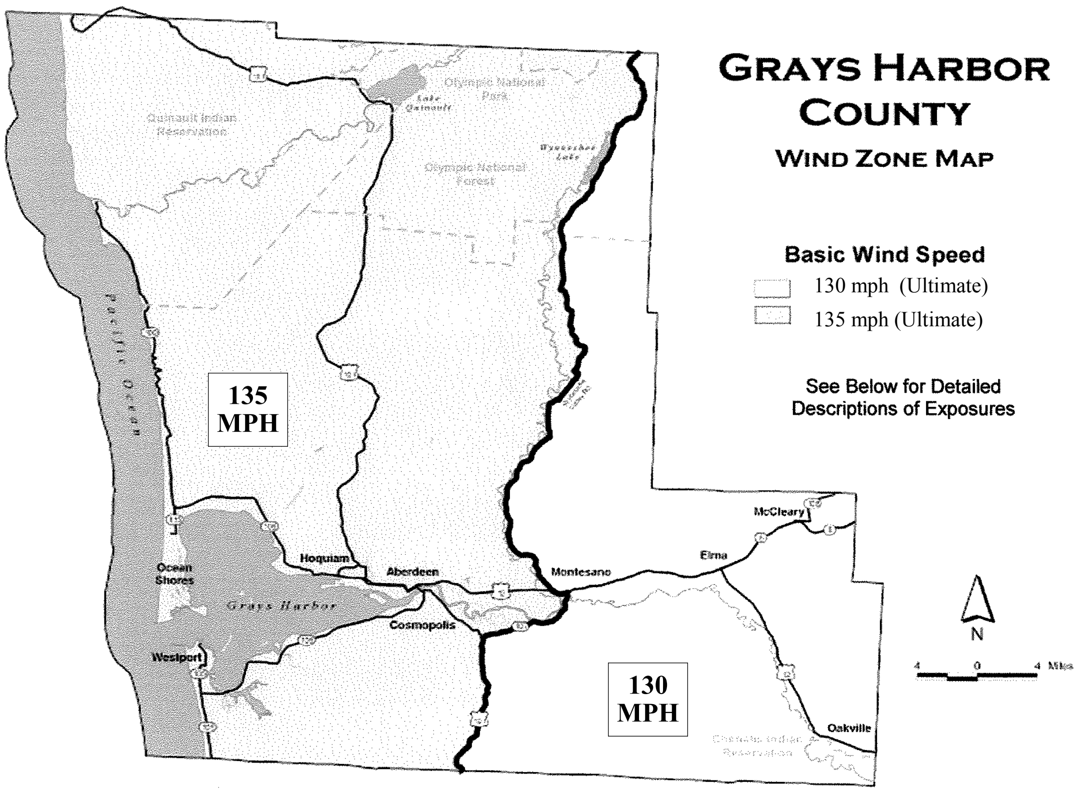
Nail all sheathing within 4 feet of wall corners w/ 8d nails @ 6" o/c edges, 12" o/c field.

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.

WIND ZONE MAP



BASIC WIND SPEED. Three-second gust speed at 33 feet (10,058 mm) above the ground in Exposure C.

1609.4 Exposure Category.
For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.

1609.4.1 Wind directions and sectors. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees (0.79 rad) either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 1609.4.2 and 1609.4.3 and the exposure resulting in the highest wind loads shall be used to represent winds from that direction.

1609.4.2 Surface roughness categories. A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

Surface Roughness B. Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, grasslands and all water surfaces in hurricane-prone regions.

Surface Roughness D. Flat, unobstructed areas and water surfaces outside hurricane-prone regions. This category includes smooth mud flats, salt flats and unbroken ice.

1609.4.3 Exposure categories. An exposure category shall be determined in accordance with the following:

Exposure B. Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or+ 20 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 30 feet (9144 mm), the upwind distance is permitted to be reduced to 1,500 feet (457 m).

Exposure C. Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall extend

SEISMIC ZONE MAP

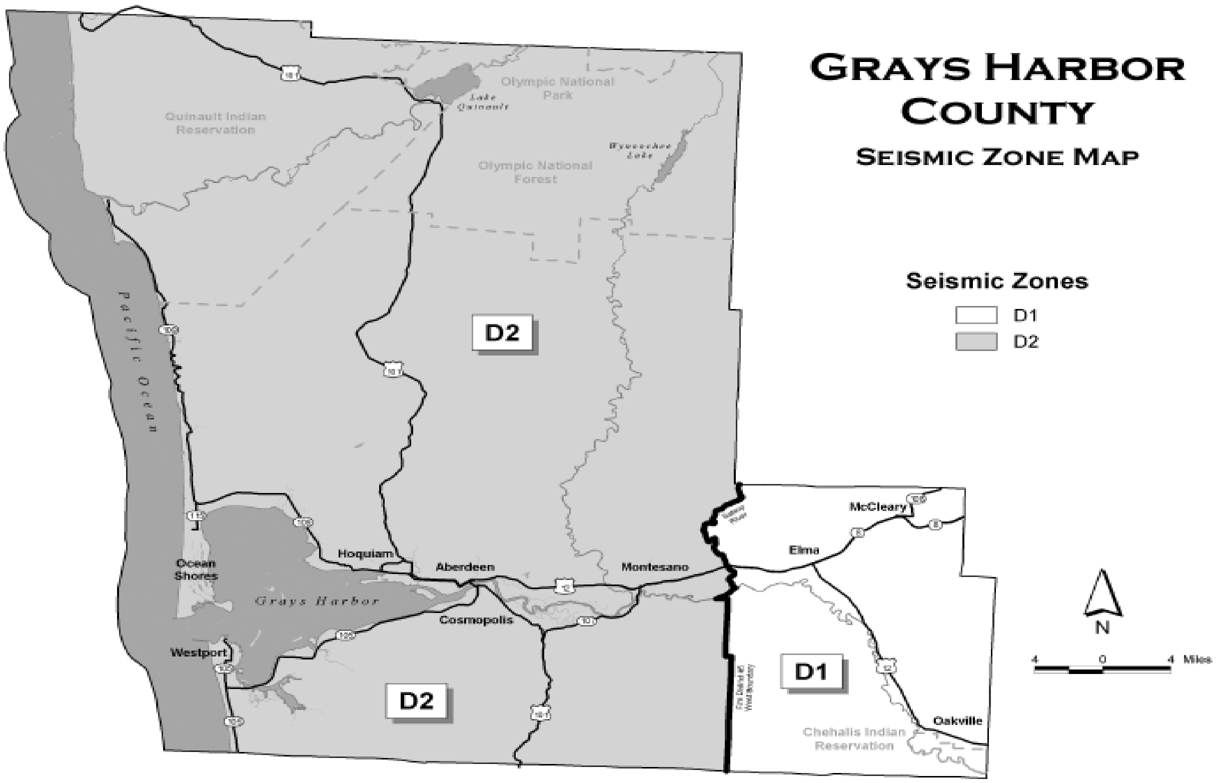


TABLE R301.2(1)
CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA

GROUND SNOW LOAD	ULT. WIND SPEED[d] (mph)	SEISMIC DESIGN CATEGORY[f]	SUBJECT TO DAMAGE FROM			WINTER DESIGN TEMP[e]	ICE BARRIER UNDERLAYMENT REQUIRED[h]	FLOOD HAZARDS[g]	AIR FREEZING INDEX[i]	MEAN ANNUAL TEMP[j]
			Weathering[a]	Frost line depth[b]	Termite[c]					
25 psf	130/135	D1/D2	Moderate	12”	Slight/Moderate	24°	NO	8/17/81 9/29/86	250	50°

For SI: 1 pound per square foot = 0.0479 kPa, 1 mile per hour = 0.447 m/s.

a. Weathering may require a higher strength concrete or grade of masonry than necessary to satisfy the structural requirements of this code. The weathering column shall be filled in with the weathering index (i.e. “negligible”, “moderate” or “severe”) for concrete as determined from the Weathering Probability Map [Figure R301.2 (3)]. The grade of masonry units shall be determined from ASTM C 34, C 55, C 62, C 73, C 90, C 129, C 145, C 216 or C 652.
b. The frost line depth may require deeper footings than indicated in Figure R403.1 (1). The jurisdiction shall fill in the frost line depth column with the minimum depth of footing below finish grade.
c. The jurisdiction shall fill in this part of the table to indicate the need for protection depending on whether there has been a history of local subterranean termite damage.
d. The jurisdiction shall fill in this part of the table with the wind speed from the basic wind speed map [Figure R301.2 (4)]. Wind exposure category shall be determined on a site-specific basis in accordance with Section R301.2.1.4.
e. The outdoor design dry-bulb temperature shall be selected from the columns of 97½-percent values for winter from Appendix D of the *International Plumbing Code.* Deviations from the Appendix D temperatures shall be permitted to reflect local climates or local weather experience as determined by the building official.
f. The jurisdiction shall fill in this part of the table with the seismic design category determined from Section R301.2.2.1.
g. The jurisdiction shall fill in this part of the table with; (a) the date of the jurisdiction’
h. s entry into the National Flood Insurance Program (date of adoption of the first code or ordinance for management of flood hazard areas). (b) the date(s) of the currently effective FIRM and FBFM, or other flood hazard map adopted by the community, as may be amended.
h. In accordance with Sections R905.2.7.1, R905.4.3.1, R905.5.3.1, R905.6.3.1, R905.7.3.1 and R905.8.3.1, where there has been a history of local damage from the effects of ice damming, the jurisdiction shall fill in this part of the table with “YES”. Otherwise, the jurisdiction shall fill in this part of the table with “NO”.
i. The jurisdiction shall fill in this part of the table with the 100-year return period air freezing index (BF-days) from Figure R403.3 (2) or from the 100-year (99%) value on the National Climatic Data Center data table “Air Freezing Index - USA Method (Base 32° Fahrenheit)” at www.ncdc.noaa.gov/fpsf.html.
j. The jurisdiction shall fill in this part of the table with the mean annual temperature from the National Climatic Data Center data table “Air Freezing Index-USA Method (Base 32°Fahrenheit)” at www.ncdc.noaa.gov/fpsf.html

Grays Harbor County
Planning & Building Division
Public Services Department
100 W Broadway Suite 31
Montesano, WA 98563
360-249-5579
360-249-3203 (fax)
pbd@co.grays-harbor.wa.us
www.co.grays-harbor.wa.us

Grays Harbor County Assessor's Office
Online Parcel Database
Assessment Information




Show Map   GeoData Viewer

Parcel 201222330050
Situs Address 03019 OCEAN BEACH RD P.BE
Legal Description TAX 1

Owner	ALOHA SELF STORAGE INC	**File Updated**	7/27/2015 02:05
Address	PO BOX 401 PACIFIC BEACH, WA 98571	**Location**	T 20 R 12 Sec 22

Certified Values:	**Land**	**Building**	**Combined**
	$28,392.00	$641,745.00	$670,137.00

Year Built	2007	**Tax Code**	064F08H2
Building Type	COMMERCIAL	**School District**	064
Style	1-STORY	**Voting Precinct**	032
Quality	AVERAGE	**Total Acres**	5.07
		Fire Patrol Acres	0
(pdf) **Land Use**	69 - Miscellaneous Services		

	Square Feet	**Type**
Lot	0	
Building SF	8734	
Percentage Complete	100%	
Basement SF	0	
Finished Basement SF	0	
Foundation	C/C	
Porch 1 SF	0	0
Porch 2 SF	0	0
Garage 1 SF	0	
Garage 2 SF	0	
Carport SF	0	0

Date Of Sale	**Excise No**	**Price**	**Instr.**	**Type**
12/30/2003	E165034	$12,500.00	WD	IL
6/17/2008	E190924	$0.00	QD	