MEDEEK ENGINEERING INC.
3050 State Route 109
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ENGINEERING REPORT

STRUCTURAL REVIEW

October 30, 2015

JOB NUMBER: **2015-048**

PLAN NUMBER: BARTH RESIDENCE

CUSTOMER: GEORGE BARTH

LOCATION: 164 OCTOPUS AVE. NE, OCEAN SHORES WA 98569

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 164 Octopus Ave. NE, Ocean Shores WA 98569.

Job#: 2015-048

ENGINEERING REPORT: STRUCTURAL REVIEW

Customer: George Barth
Location: 164 Octopus Ave. NE, Ocean Shores WA 98569
Engr: Nathaniel P. Wilkerson
Date: 30-Oct-15

CODES

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

DESIGN CRITERIA SUMMARY

Ground Snow Load	25.0	PSF
Frost Line Depth	12.0	IN
Occupancy Classification	R	
Risk Category	II	
Snow Importance Factor (I_s)	1.0	
Wind Speed (ultimate)	155.0	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	6.00	:12
Roof Eve Height	11.000	FT
Peak Roof Height	19.500	FT
Mean Roof Height	15.250	FT
Building Length (L)	56	FT
Building Width (B)	55	FT
Latitude	46.9869	N
Longitude	124.1539	W
Elevation:	18.0	FT

SEISMIC

S_{DS}		0.979	g
S_{D1}		0.735	g
S_{MS}		1.468	g
S_{M1}		1.103	g
S_S		1.468	g
S_1		0.735	g
F_a		1.000	
F_v		1.500	
Roof Diaphragm Height (h_n)*		15.25	FT
Fundamental Period (T_a)	$T_a = C_t h_n^x$ =	0.154	sec.
T_0		0.150	sec.
T_S		0.751	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 $T_a \le T_L$)	$C_s = \dfrac{S_{D1}}{T_a\left(\dfrac{R}{I_e}\right)}$ =	0.733	

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

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \quad = \quad 0.039$$

if $S_1 \ge 0.6$g:

$$C_s = \frac{0.5 S_1}{\left(\dfrac{R}{I_e}\right)} \quad = \quad 0.057$$

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

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Resources Seismic Wind Snow 2x6 Framing Trusses Contractors CAD Library Forum

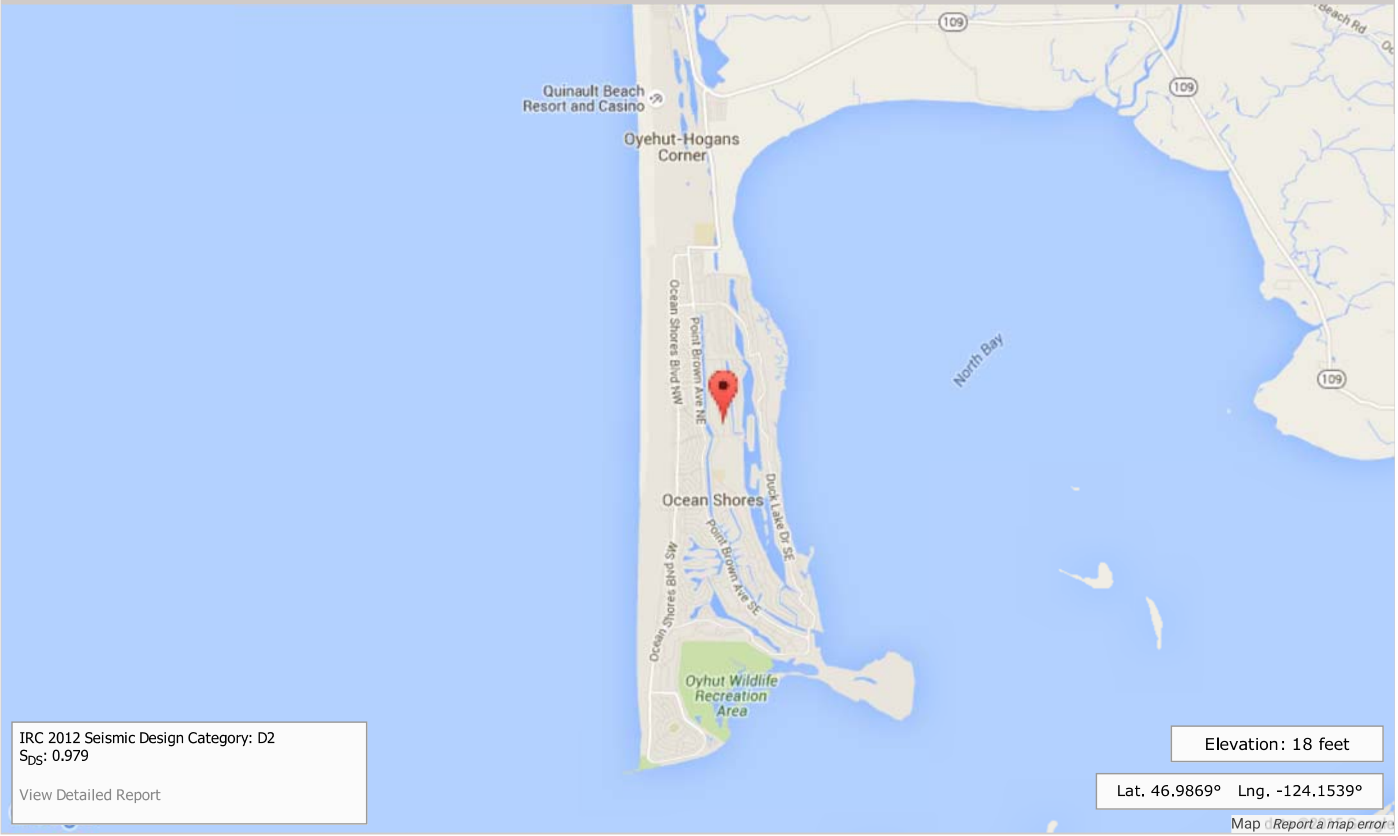
MEDEEK DESIGN

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: 164 Octopus Ave. NE City: Ocean Shores State: Washington Zip: 98535 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.

USGS Design Maps Summary Report

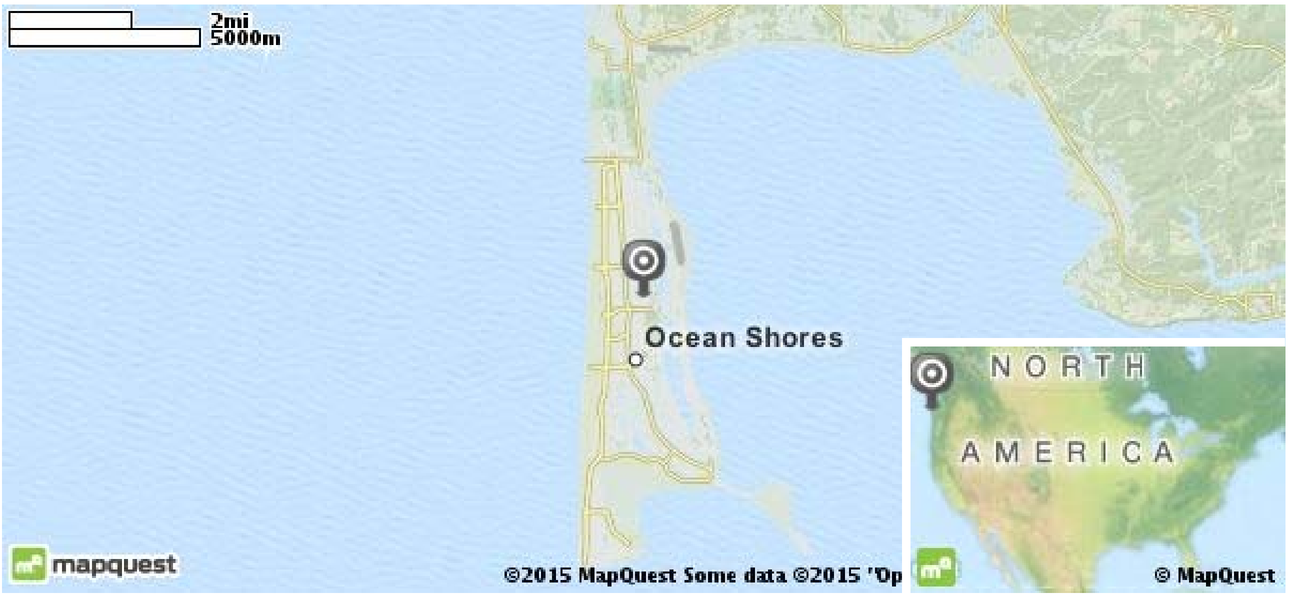
User–Specified Input

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 46.9869°N, 124.1539°W

Site Soil Classification Site Class D – "Stiff Soil"

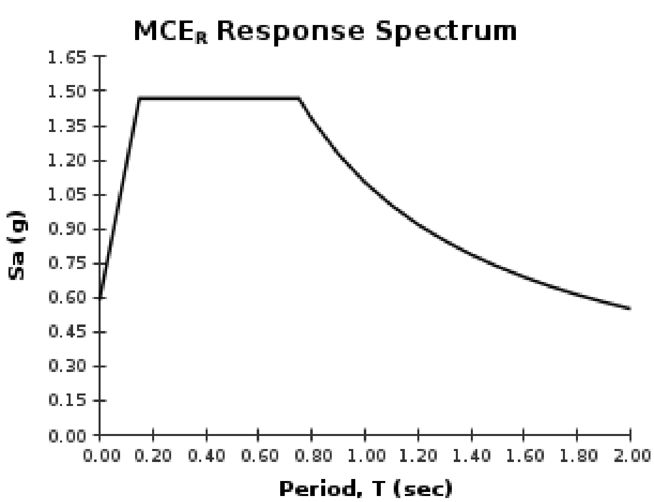
Risk Category I/II/III

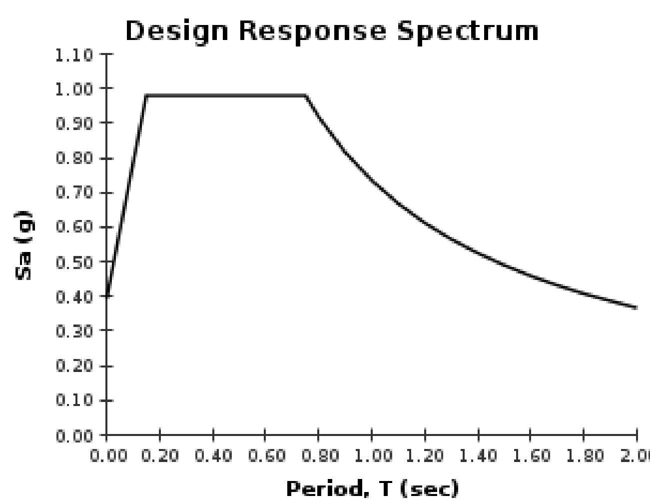


USGS–Provided Output

S_S =	1.468 g	S_{MS} =	1.468 g	S_{DS} =	0.979 g
S_1 =	0.735 g	S_{M1} =	1.102 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:	6 /12
Risk Category:	II
Eave-to-Ridge (W):	7.333 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	Common 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) = 1.12 (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): 7.8 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

Subject	Customer	Location	Job No.
Snow Loads	George Barth	164 Octopus Ave. NE Ocean Shores WA 98569	2015-048

Engr. N. Wilkerson

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Date 10/30/2015

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 (26.57°) is greater than W/50 = 0.1, 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.

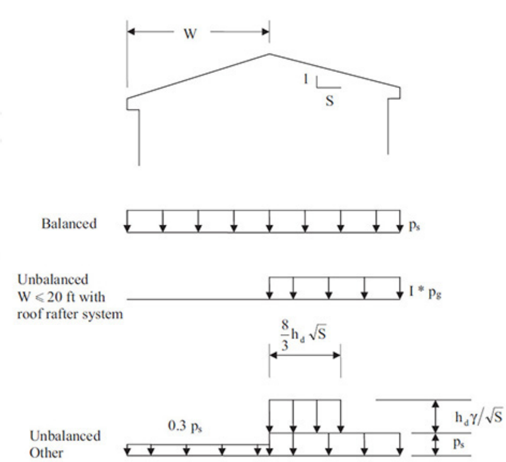
$p_s = C_s p_f = (1.00)(19.3) = 19.3$ psf

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) [if l_u < 20 ft., use l_u = 20 ft.]
$l_d = \frac{8}{3} h_d \sqrt{S}$ (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



Note: Unbalanced loads need not be considered for θ > 30.2° (7 on 12) or for θ ≤ 2.38° (1/2 on 12).

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

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

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

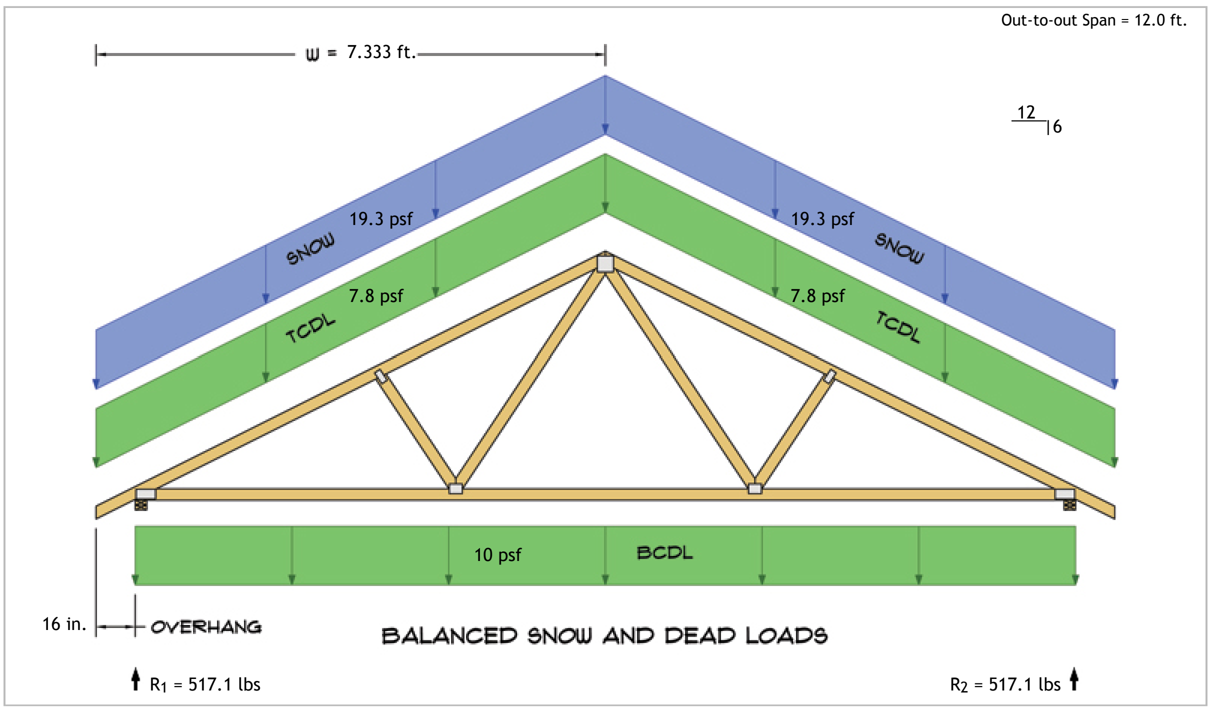
$$h_d = .43\sqrt[3]{20}\sqrt[4]{25.0+10} - 1.5 = 1.34 \text{ ft. } [l_u = 20 \text{ ft.}]$$

$$l_d = \frac{8}{3} \times 1.34 \times \sqrt{12/6} = 5.05 \text{ ft.}$$

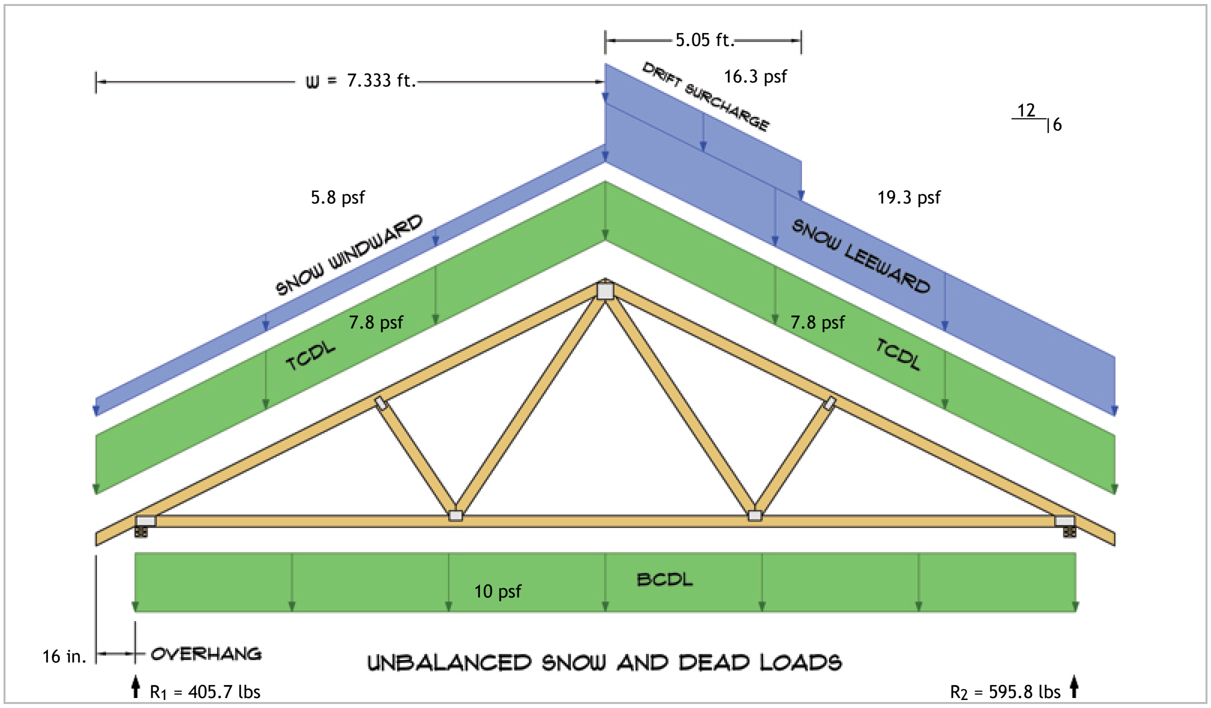
$$p_d = \frac{1.34 \times 17.25}{\sqrt{12/6}} = 16.3 \text{ psf}$$

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.

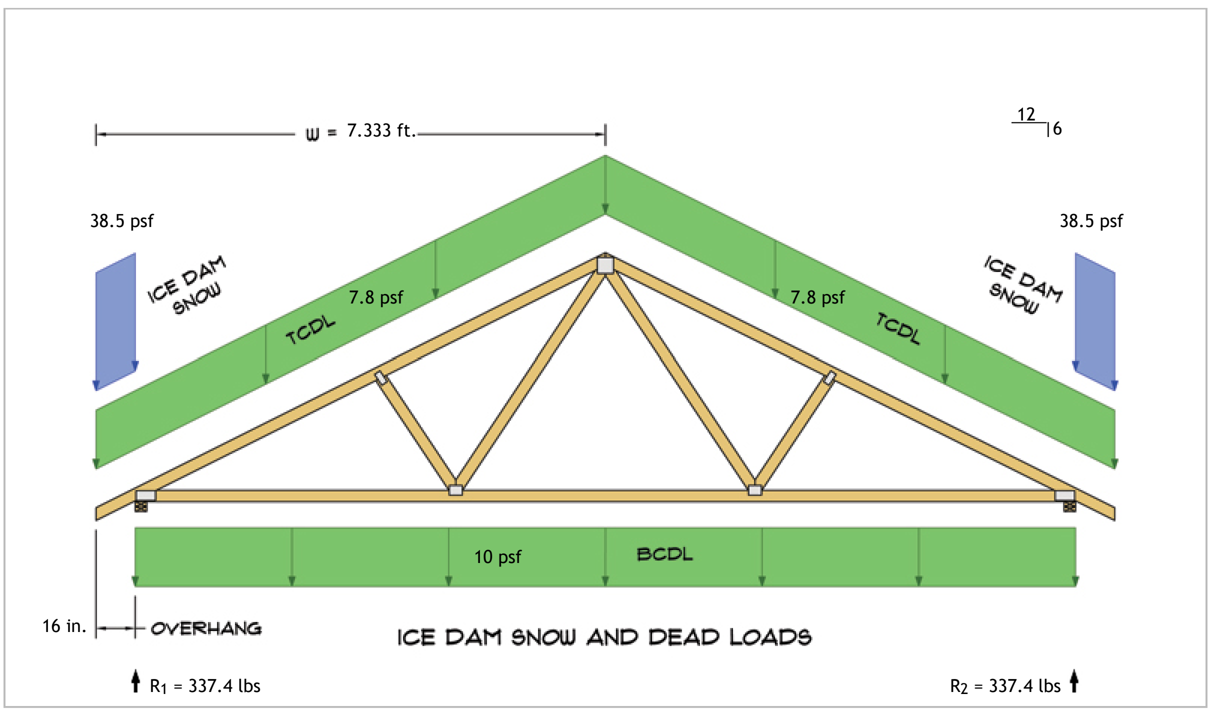
$2p_f = (2)(19.3) = 38.5$ psf



$R_1 = D + S = 234.8$ lbs $+ 282.3$ lbs
$R_2 = D + S = 234.8$ lbs $+ 282.3$ lbs



$R_1 = D + S = 234.8$ lbs + 170.9 lbs
$R_2 = D + S = 234.8$ lbs + 361.1 lbs



$R_1 = D + S = 234.8$ lbs + 102.7 lbs
$R_2 = D + S = 234.8$ lbs + 102.7 lbs

Subject	Customer	Location	Job No.
Snow Loads	George Barth	164 Octopus Ave. NE Ocean Shores WA 98569	2015-048

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Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	6 /12
Risk Category:	II
Eave-to-Ridge (W):	9.333 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	Common 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) = 1.12 (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): 7.8 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

$$p_f = 0.7C_eC_tI_sp_g = 0.7(1.00)(1.10)(1.00)(25.0) = 19.3 \text{ psf}$$

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 (26.57°) is greater than W/50 = 0.2, 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) [if l_u < 20 ft., use l_u = 20 ft.]
$l_d = \frac{8}{3}h_d\sqrt{S}$ (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

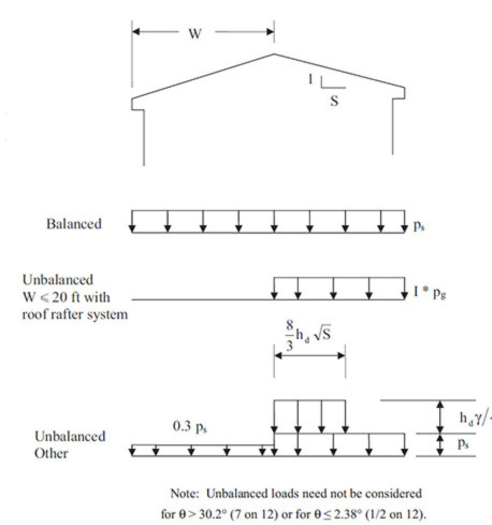


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

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

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

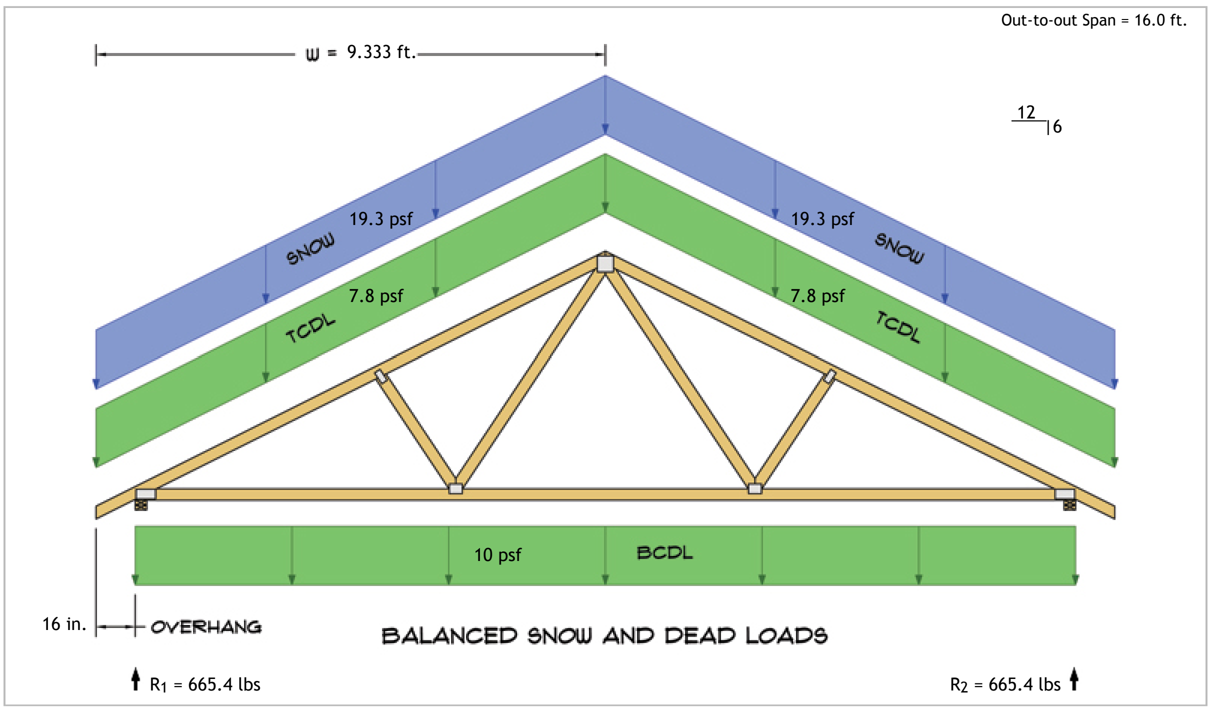
$$h_d = .43\sqrt[3]{20}\sqrt[4]{25.0+10} - 1.5 = 1.34 \text{ ft. } [l_u = 20 \text{ ft.}]$$

$$l_d = \frac{8}{3} \times 1.34 \times \sqrt{12/6} = 5.05 \text{ ft.}$$

$$p_d = \frac{1.34 \times 17.25}{\sqrt{12/6}} = 16.3 \text{ psf}$$

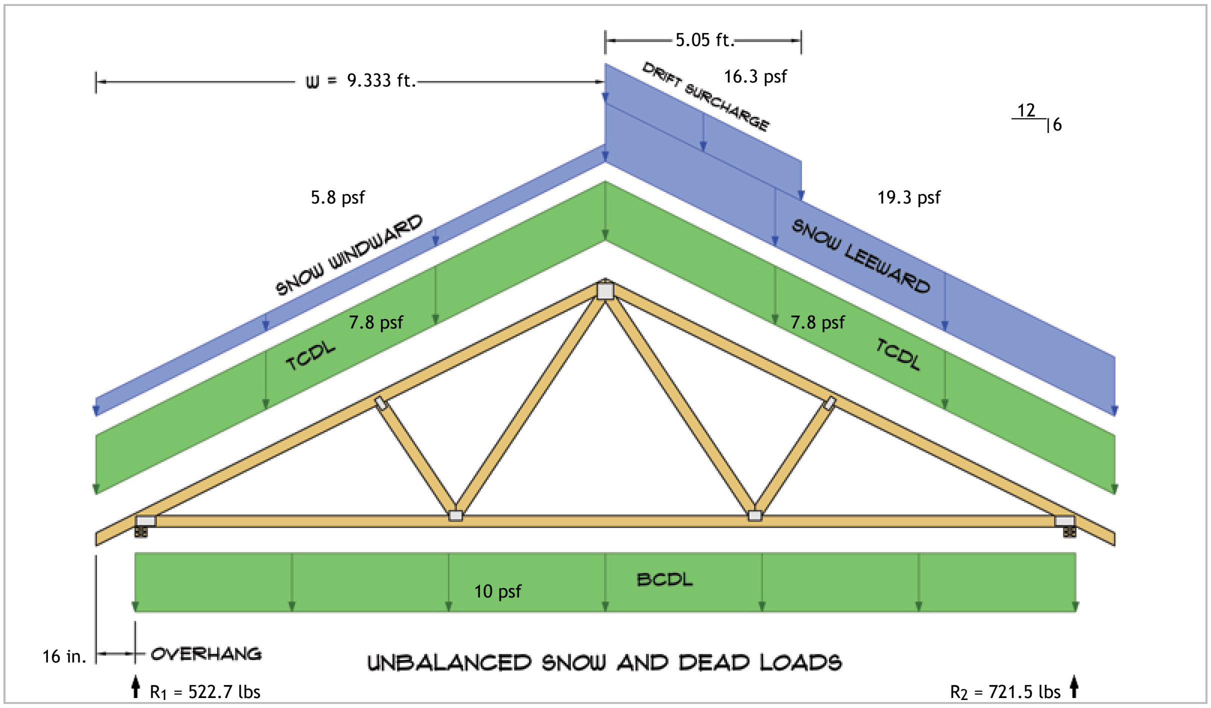
On warm roofs apply a distributed $2p_f$ snow load on all overhanging portions as per ASCE 7-10 section 7.4.5.
No other loads except dead loads shall be present on the roof when this uniformly distributed load is applied.

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

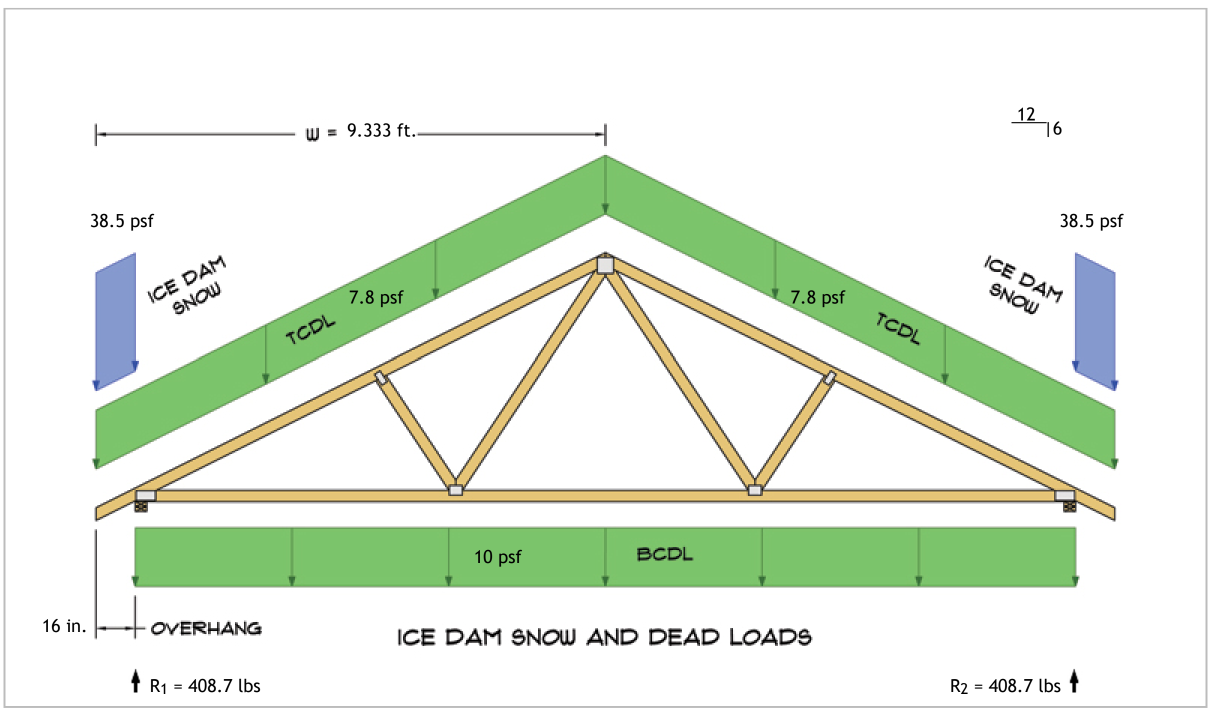


$R_1 = D + S = 306.1$ lbs + 359.3 lbs
$R_2 = D + S = 306.1$ lbs + 359.3 lbs

Subject	Customer	Location	Job No.
Snow Loads	George Barth	164 Octopus Ave. NE Ocean Shores WA 98569	2015-048



$R_1 = D + S = 306.1$ lbs + 216.6 lbs
$R_2 = D + S = 306.1$ lbs + 415.4 lbs



$R_1 = D + S = 306.1$ lbs + 102.7 lbs
$R_2 = D + S = 306.1$ lbs + 102.7 lbs

Subject	Customer	Location	Job No.
Snow Loads	George Barth	164 Octopus Ave. NE Ocean Shores WA 98569	2015-048

Engr.
N. Wilkerson

Date
10/30/2015

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Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	6 /12
Risk Category:	II
Eave-to-Ridge (W):	8.333 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	Common 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) = 1.12 (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): 7.8 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 (26.57°) is greater than W/50 = 0.2, 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) [if l_u < 20 ft., use l_u = 20 ft.]
$l_d = \frac{8}{3}h_d\sqrt{S}$ (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

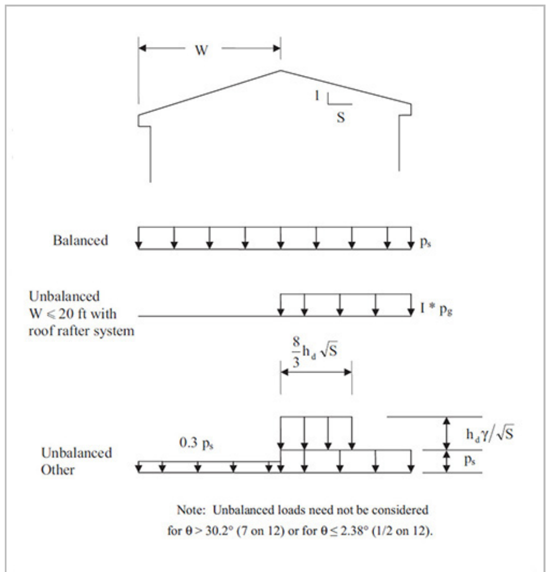


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

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

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

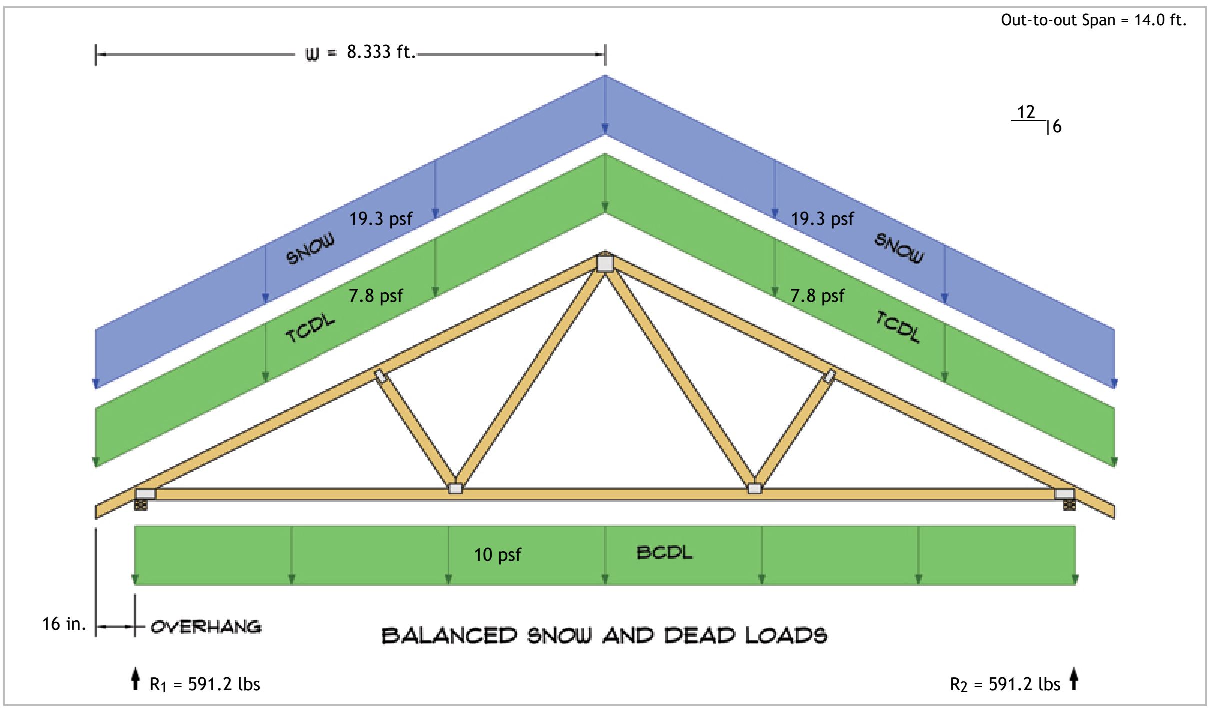
$h_d = .43\sqrt[3]{20}\sqrt[4]{25.0+10} - 1.5 = 1.34$ ft. [$l_u = 20$ ft.]

$l_d = \frac{8}{3} \times 1.34 \times \sqrt{12/6} = 5.05$ ft.

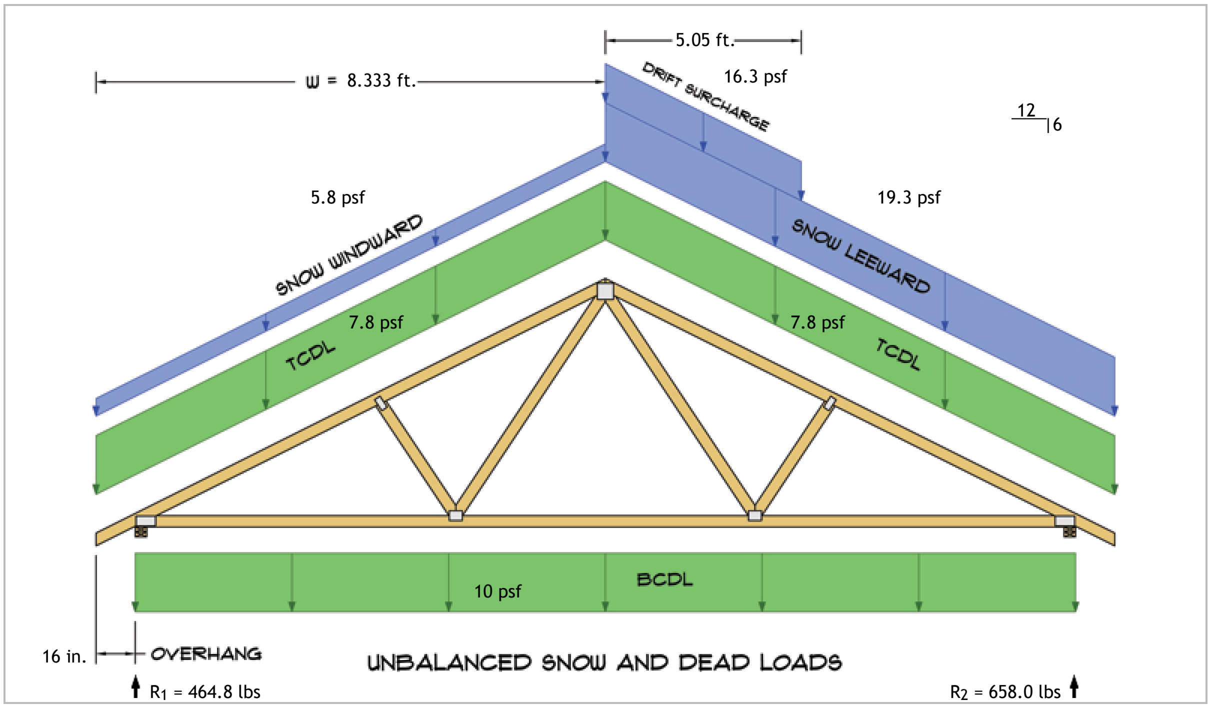
$p_d = \frac{1.34 \times 17.25}{\sqrt{12/6}} = 16.3$ psf

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.

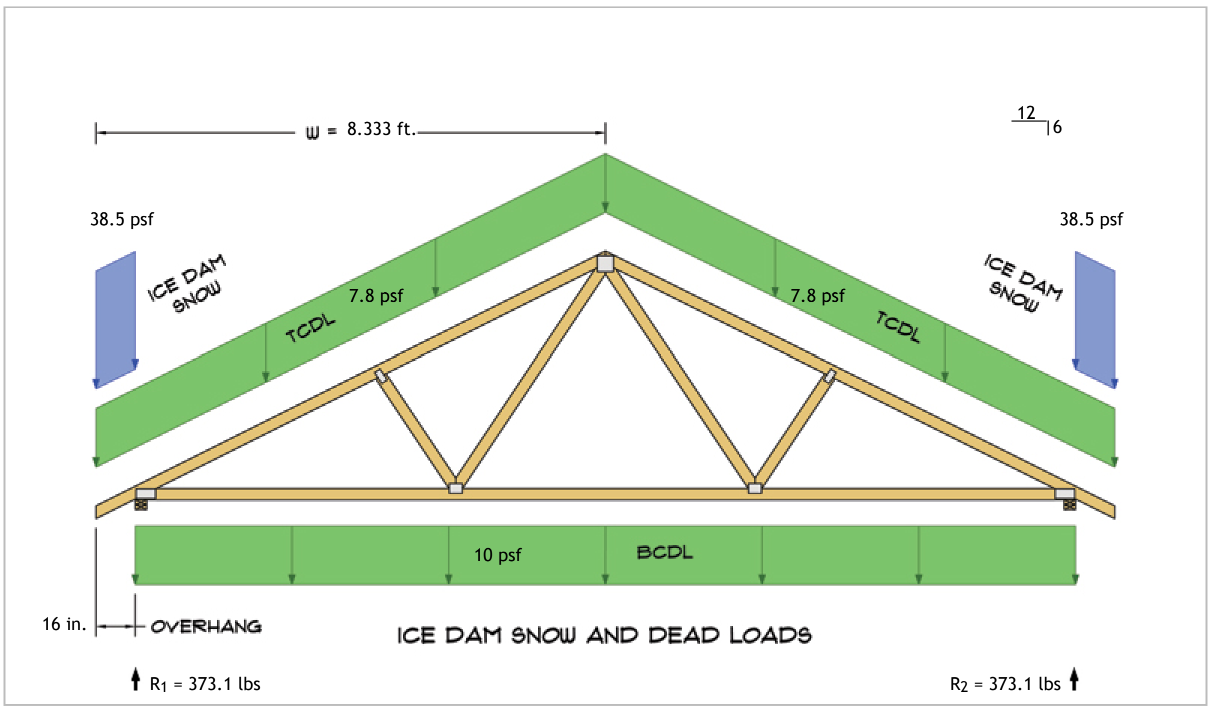
$2p_f = (2)(19.3) = 38.5$ psf



$R_1 = D + S = 270.4$ lbs + 320.8 lbs
$R_2 = D + S = 270.4$ lbs + 320.8 lbs



$R_1 = D + S = 270.4$ lbs + 194.4 lbs
$R_2 = D + S = 270.4$ lbs + 387.6 lbs



$R_1 = D + S = 270.4$ lbs + 102.7 lbs
$R_2 = D + S = 270.4$ lbs + 102.7 lbs

Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	6 /12
Risk Category:	II
Eave-to-Ridge (W):	14.333 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	Common 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) = 1.12 (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): 7.8 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 (26.57°) is greater than W/50 = 0.3, 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) [if l_u < 20 ft., use l_u = 20 ft.]
$l_d = \frac{8}{3} h_d \sqrt{S}$ (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

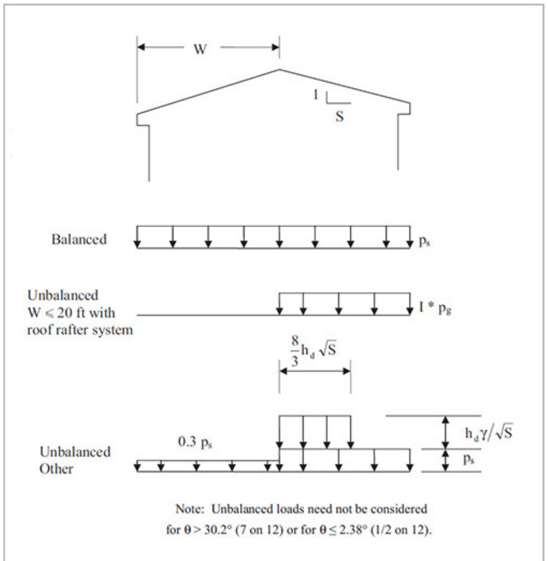


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

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

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

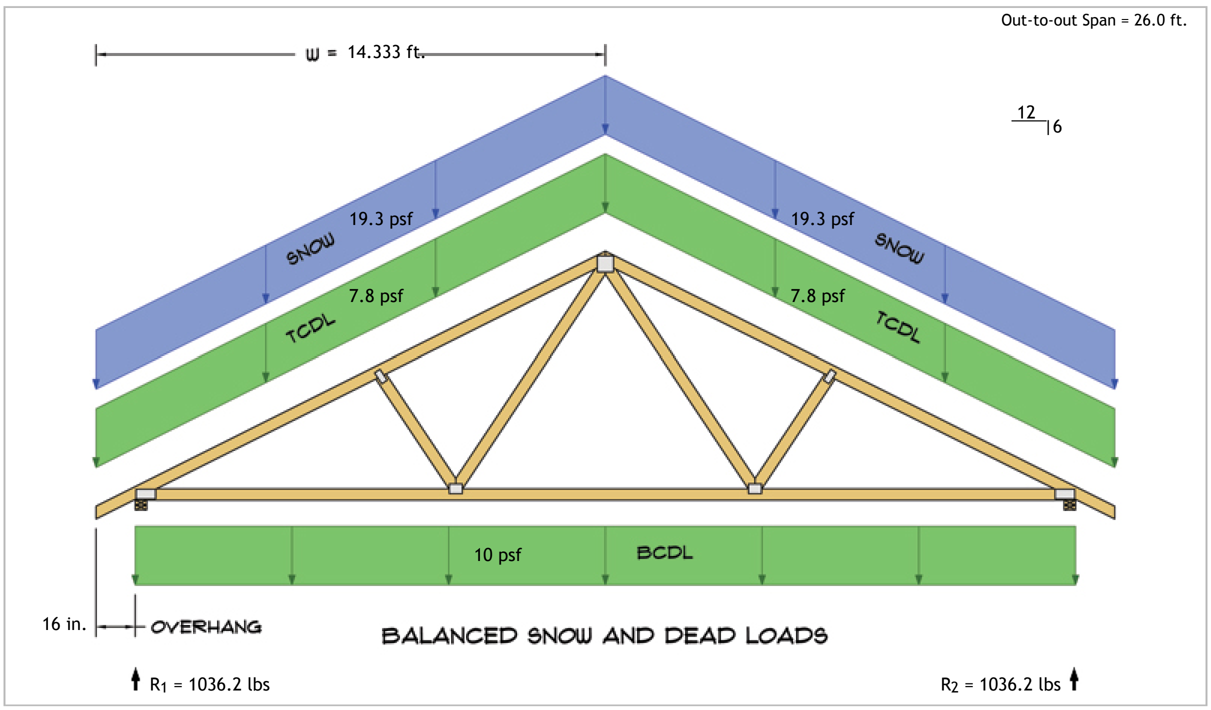
$$h_d = .43\sqrt[3]{20}\sqrt[4]{25.0+10} - 1.5 = 1.34 \text{ ft. } [l_u = 20 \text{ ft.}]$$

$$l_d = \frac{8}{3} \times 1.34 \times \sqrt{12/6} = 5.05 \text{ ft.}$$

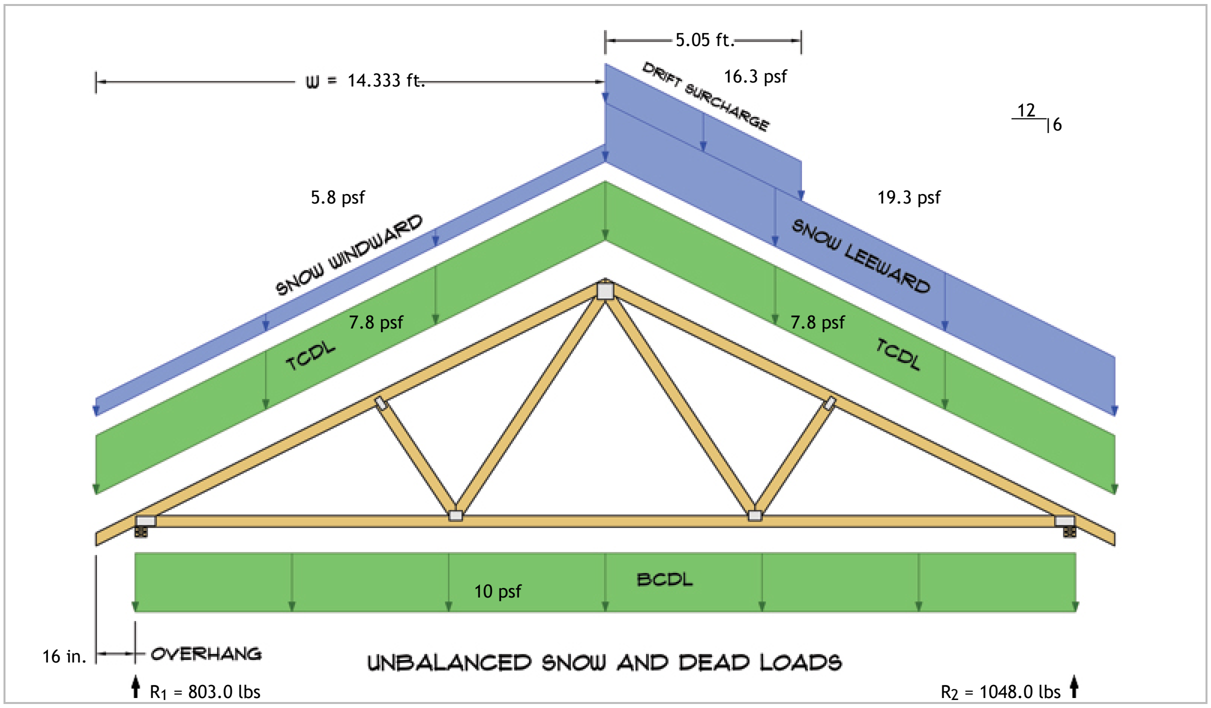
$$p_d = \frac{1.34 \times 17.25}{\sqrt{12/6}} = 16.3 \text{ psf}$$

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.

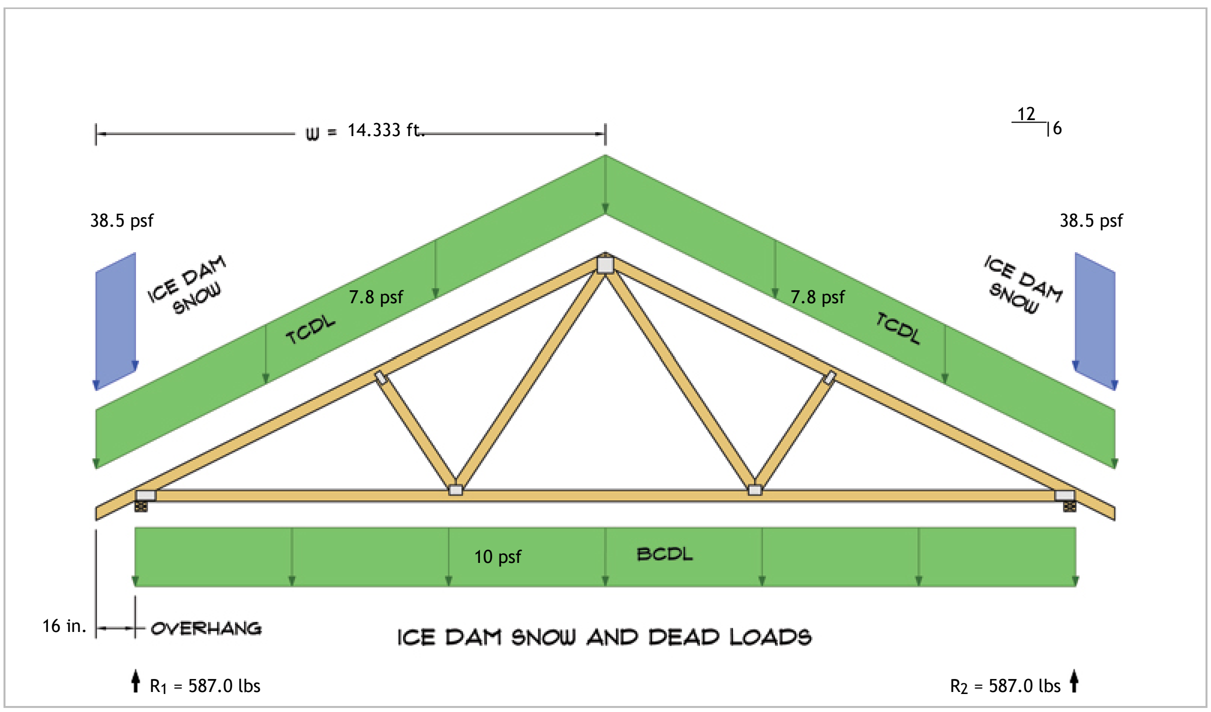
$2p_f = (2)(19.3) = 38.5$ psf



$R_1 = D + S = 484.3$ lbs $+ 551.8$ lbs
$R_2 = D + S = 484.3$ lbs $+ 551.8$ lbs



$R_1 = D + S = 484.3$ lbs + 318.7 lbs
$R_2 = D + S = 484.3$ lbs + 563.6 lbs



$R_1 = D + S = 484.3$ lbs + 102.7 lbs
$R_2 = D + S = 484.3$ lbs + 102.7 lbs

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Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	6 /12
Risk Category:	II
Eave-to-Ridge (W):	17.0833 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.10
Roof Surface:	Asphalt Shingles
Roof System:	Common 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) = 1.12 (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): 7.8 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 (26.57°) is greater than W/50 = 0.3, 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) [if l_u < 20 ft., use l_u = 20 ft.]
$l_d = \frac{8}{3} h_d \sqrt{S}$ (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

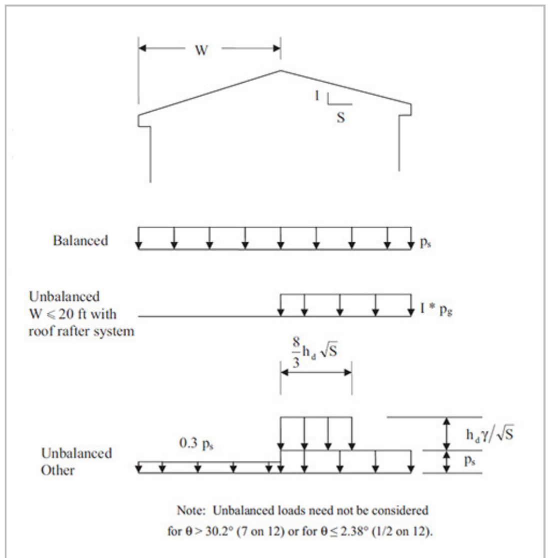


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

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

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

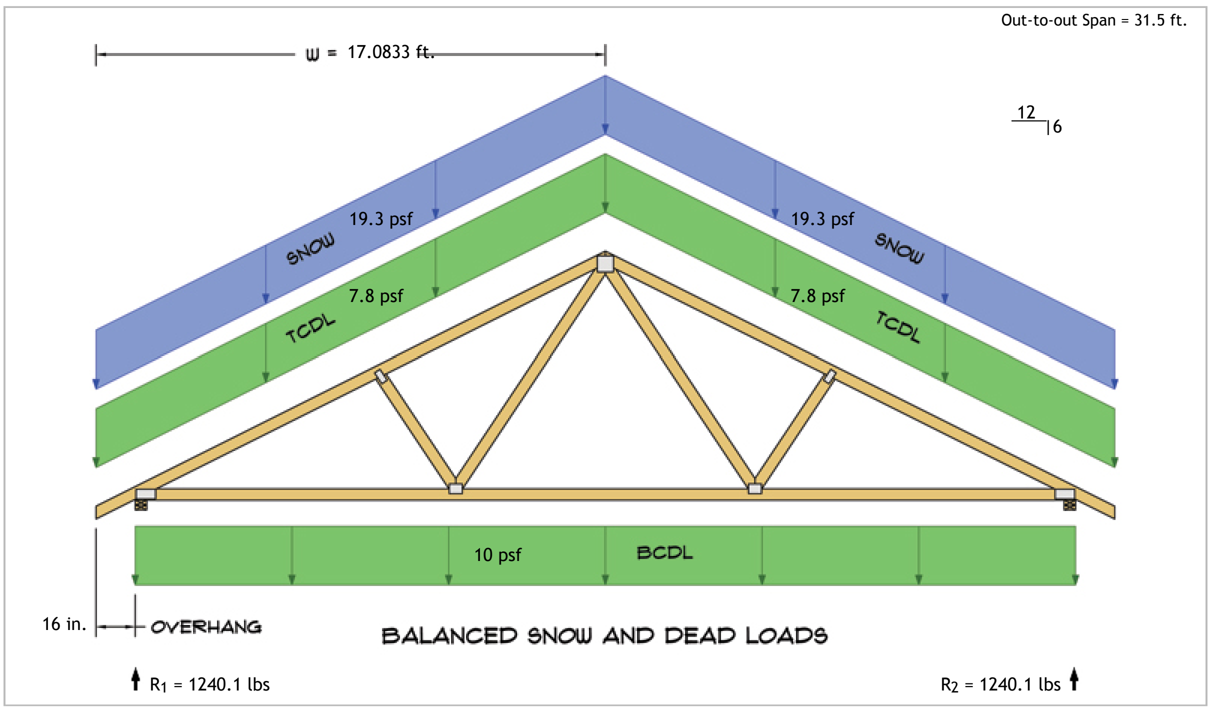
$$h_d = .43\sqrt[3]{20}\sqrt[4]{25.0+10} - 1.5 = 1.34 \text{ ft.} \quad [l_u = 20 \text{ ft.}]$$

$$l_d = \frac{8}{3} \times 1.34 \times \sqrt{12/6} = 5.05 \text{ ft.}$$

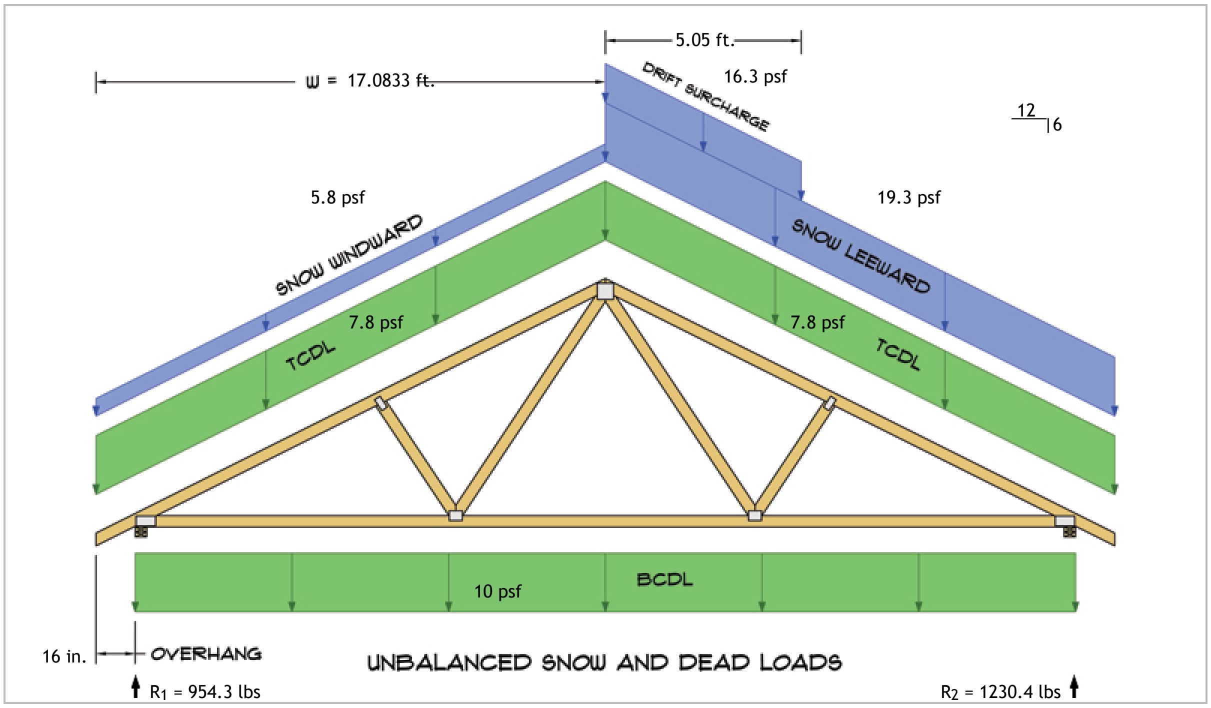
$$p_d = \frac{1.34 \times 17.25}{\sqrt{12/6}} = 16.3 \text{ psf}$$

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.

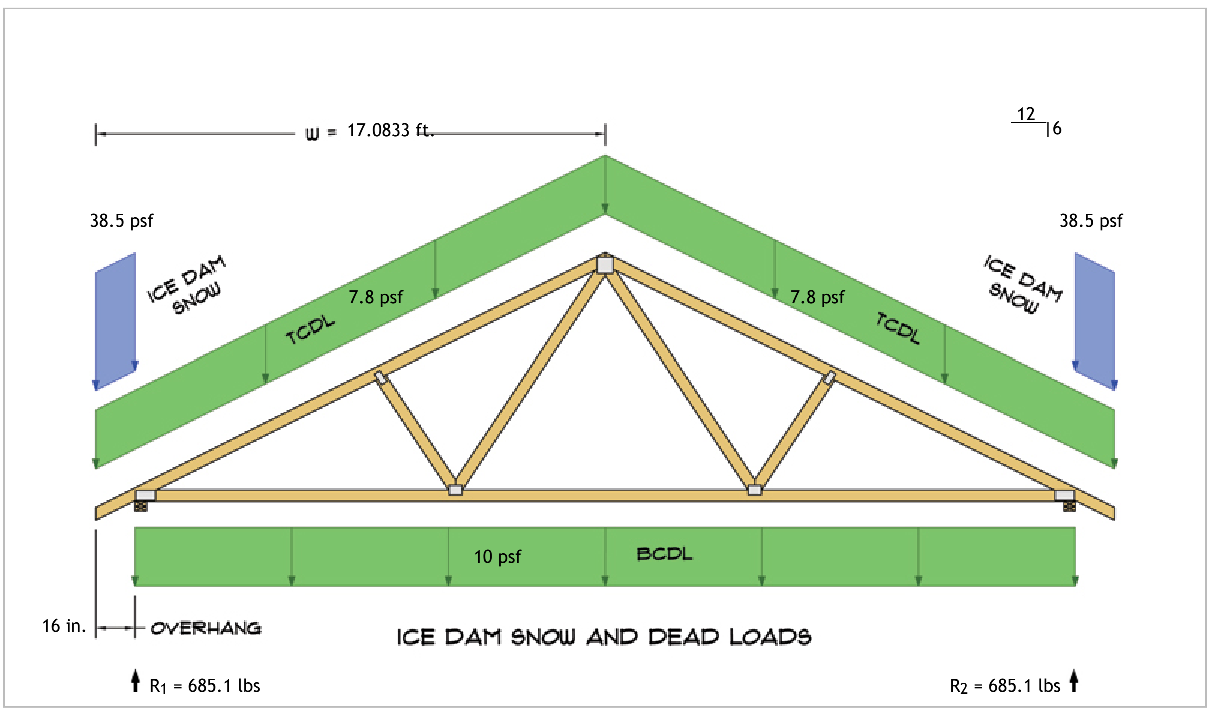
$2p_f = (2)(19.3) = 38.5$ psf



$R_1 = D + S = 582.4$ lbs $+ 657.7$ lbs
$R_2 = D + S = 582.4$ lbs $+ 657.7$ lbs



$R_1 = D + S = 582.4$ lbs + 371.9 lbs
$R_2 = D + S = 582.4$ lbs + 648.0 lbs



$R_1 = D + S = 582.4$ lbs + 102.7 lbs
$R_2 = D + S = 582.4$ lbs + 102.7 lbs

Subject	Customer	Location	Job No.
Snow Loads	George Barth	164 Octopus Ave. NE Ocean Shores WA 98569	2015-048

Engr.
N. Wilkerson

Date
10/30/2015

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Rev.
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WIND (MWFRS)

Wind Analysis Method	Analytic Directional Procedure		ASCE 7-10 Fig. 27.4-1
Basic Wind Speed (ultimate)		155.00 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		6.00 :12	26.57 DEG
Roof Eve Height		11.000 FT	
Peak Roof Height		19.500 FT	α = 9.5
Mean Roof Height		15.250 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 =	11.00	FT	0.849	44.38
h =	15.25	FT	0.852	**44.53**
z =	15	FT	0.849	44.38
z =	20	FT	0.902	47.15
z =	25	FT	0.945	49.42
z =	30	FT	0.982	51.35

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 = 55 L/B = 0.98
B = 56 h/L = 0.28

	z (ft)	q_z (psf)	Cp	qGC_p	Design Pressure (psf) (+GCpi)	(-GCpi)
Windward Wall	15	44.38	0.80	30.18	**22.16**	**38.19**
	20	47.15	0.80	32.06	**24.05**	**40.08**
	25	49.42	0.80	33.60	**25.59**	**41.62**
	30	51.35	0.80	34.92	**26.90**	**42.93**
Leeward Wall	15.25	44.53	-0.50	-18.93	**-26.94**	**-10.91**
Side Wall	15.25	44.53	-0.70	-26.50	**-34.51**	**-18.48**
Windward Roof (Positive)	15.25	44.53	0.29	10.94	**2.93**	**18.96**
Windward Roof (Negative)	15.25	44.53	-0.21	-7.85	**-15.87**	**0.16**
Leeward Roof	15.25	44.53	-0.60	-22.71	**-30.73**	**-14.70**
Ridge Parallel Roof	(0 to h/2)	44.53	-0.90	-34.07	**-42.08**	**-26.05**
	(h/2 to h)	44.53	-0.90	-34.07	**-42.08**	**-26.05**
	(h to 2h)	44.53	-0.50	-18.93	**-26.94**	**-10.91**
	(>h2)	44.53	-0.30	-11.36	**-19.37**	**-3.34**

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

L = 56 L/B = 1.02
B = 55 h/L = 0.27

	z (ft)	q_z (psf)	Cp	qGC_p	Design Pressure (psf) (+GCpi)	(-GCpi)
Windward Wall	15	44.38	0.80	30.18	**22.16**	**38.19**
	20	47.15	0.80	32.06	**24.05**	**40.08**
	25	49.42	0.80	33.60	**25.59**	**41.62**
	30	51.35	0.80	34.92	**26.90**	**42.93**
Leeward Wall	15.25	44.53	-0.50	-18.79	**-26.80**	**-10.77**
Side Wall	15.25	44.53	-0.70	-26.50	**-34.51**	**-18.48**
Windward Roof (Positive)	15.25	44.53	0.29	11.02	**3.00**	**19.03**
Windward Roof (Negative)	15.25	44.53	-0.21	-7.80	**-15.82**	**0.21**
Leeward Roof	15.25	44.53	-0.60	-22.71	**-30.73**	**-14.70**
Ridge Parallel Roof	(0 to h/2)	44.53	-0.90	-34.07	**-42.08**	**-26.05**
	(h/2 to h)	44.53	-0.90	-34.07	**-42.08**	**-26.05**
	(h to 2h)	44.53	-0.50	-18.93	**-26.94**	**-10.91**
	(>h2)	44.53	-0.30	-11.36	**-19.37**	**-3.34**

$p = qGC_p$

Overhangs:	z (ft)	q_z (psf)	Cp	qGC_p
Windward Overhang	11.000	44.38	0.80	**30.18**

Job#: **2015-048**

WIND (C&C)

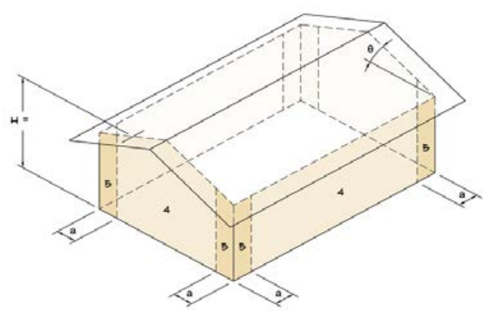
Wind Analysis Method			Part 1: Low Rise Buildings		
Basic Wind Speed (ultimate)		155.00	MPH		
Topography Factor	K_{zt} =	1.00		ASCE 7-10 Fig. 26.8-1	
Directionality Factor	K_d =	0.85		ASCE 7-10 Fig. 26.6-1	
Internal Pressure Coefficients	(GC_{pi}) =	0.18	-0.18	ASCE 7-10 Table 26.11-1	
Roof Pitch		6.00	:12	26.57 DEG	
Roof Eve Height		11.000	FT		
Peak Roof Height		19.500	FT	α =	9.5
Mean Roof Height		15.250	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 =	15.25	FT	0.852	**44.53**

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

Component	Span Length (ft.)	Width (ft.)	Trib. Area	Eff. Area
Stud	9	1.33	11.97	27.00
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	27.00	0.92	-1.02	0.92	-1.25
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	27.00	**49.16**	**-53.61**	**49.16**	**-63.58**
Panel	32.00	**48.58**	**-53.03**	**48.58**	**-62.42**
A ≤ 10 ft^2	10.00	52.55	-57.00	52.55	-70.36
A = 20 ft^2	20.00	50.18	-54.63	50.18	-65.63
A = 50 ft^2	50.00	47.05	-51.51	47.05	-59.37
A = 100 ft^2	100.00	44.68	-49.14	44.68	-54.63
A = 200 ft^2	200.00	42.32	-46.77	42.32	-49.90
A ≥ 500 ft^2	500.00	39.19	-43.64	39.19	-43.64

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

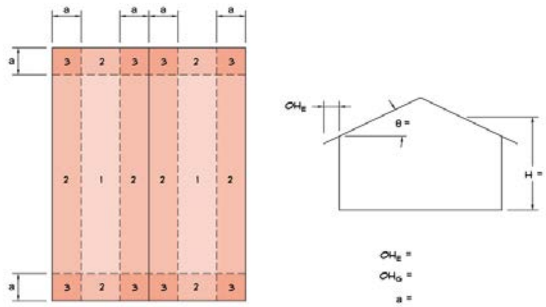
Min. Pressure: The design wind pressure for C&C shall not be less than 16 psf acting in either direction normal to the surface.

Job#: **2015-048**

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

Roof Components

Component	Span Length (ft.)	Width (ft.)	Trib. Area	Eff. Area
Truss/Rafter	14	2	28.00	65.33
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	65.33	0.34	-0.82	0.34	-1.29	0.34	-2.11
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	65.33	**23.02**	**-44.47**	**23.02**	**-65.57**	**23.02**	**-102.02**
Panel	32.00	**25.78**	**-45.85**	**25.78**	**-72.47**	**25.78**	**-110.30**
A ≤ 10 ft^2	10.00	30.28	-48.10	30.28	-83.72	30.28	-123.80
A = 20 ft^2	20.00	27.60	-46.76	27.60	-77.02	27.60	-115.76
A = 50 ft^2	50.00	24.06	-44.98	24.06	-68.16	24.06	-105.13
A = 100 ft^2	100.00	21.38	-43.64	21.38	-61.46	21.38	-97.08

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	65.33	0.34	-0.82	0.34	-2.20	0.34	-2.72
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	65.33	**23.02**	**-44.47**	**23.02**	**-105.99**	**23.02**	**-129.23**
Panel	32.00	**25.78**	**-45.85**	**25.78**	**-105.99**	**25.78**	**-145.79**
A ≤ 10 ft^2	10.00	30.28	-48.10	30.28	-105.99	30.28	-172.79
A = 20 ft^2	20.00	27.60	-46.76	27.60	-105.99	27.60	-156.70
A = 50 ft^2	50.00	24.06	-44.98	24.06	-105.99	24.06	-135.44
A = 100 ft^2	100.00	21.38	-43.64	21.38	-105.99	21.38	-119.35

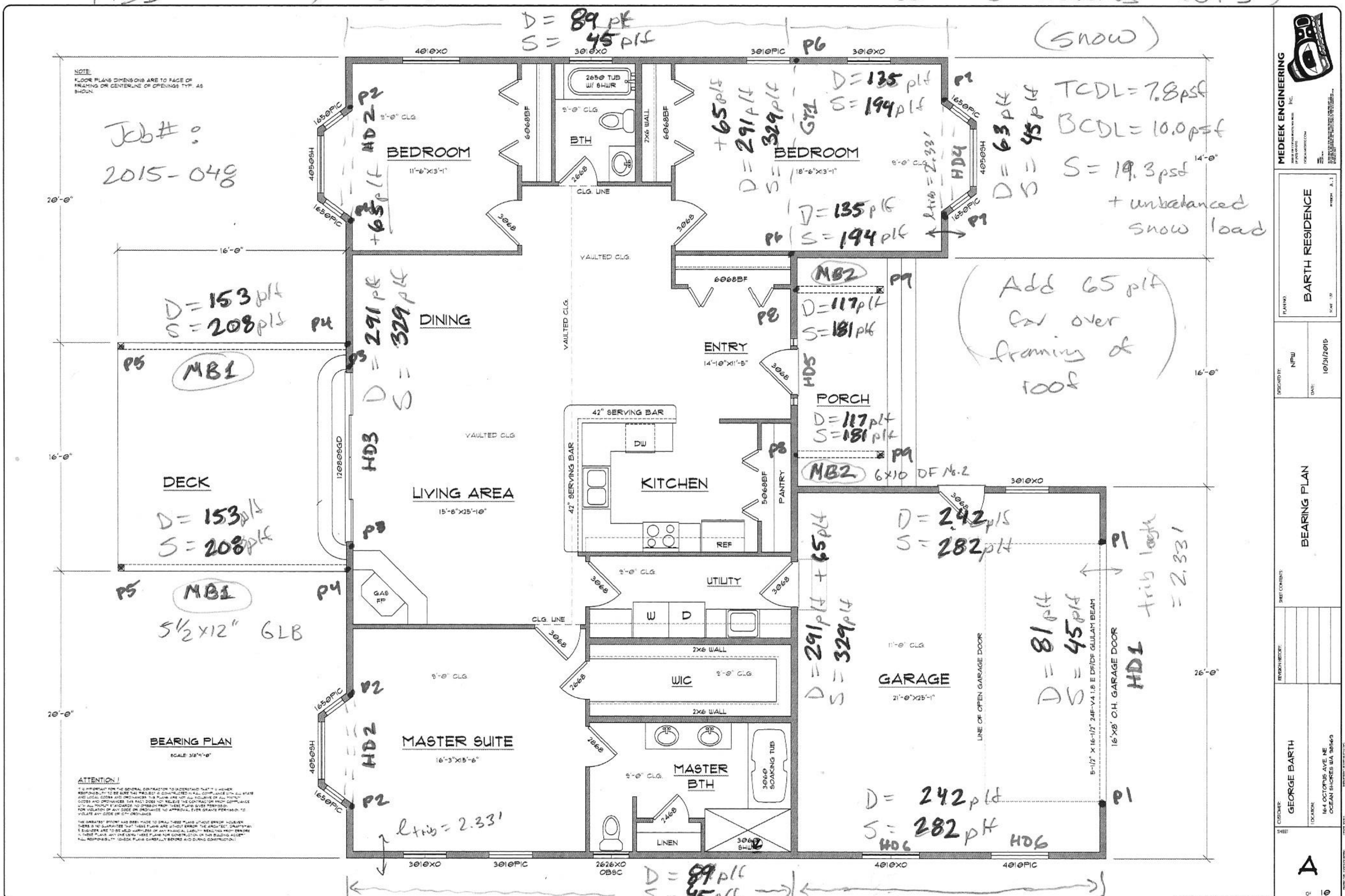
Width of Zones 2,3 and 5

smaller of:	0.1 x	55.00 =	5.50 ft	**(controls)**	a = 5.5 ft
	0.4 x	15.25 =	6.10 ft		
not less than:	0.04 x	55.00 =	2.20 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.

MEI Oct 31, 2015 NAW ASCE 7-10 Load Case #3 Governs (D+S)



Oct. 31, 2015	MEI - NPW	Job # : 2015-048

Bearing Walls @ Gable Ends

1) Master Suite

trib length:

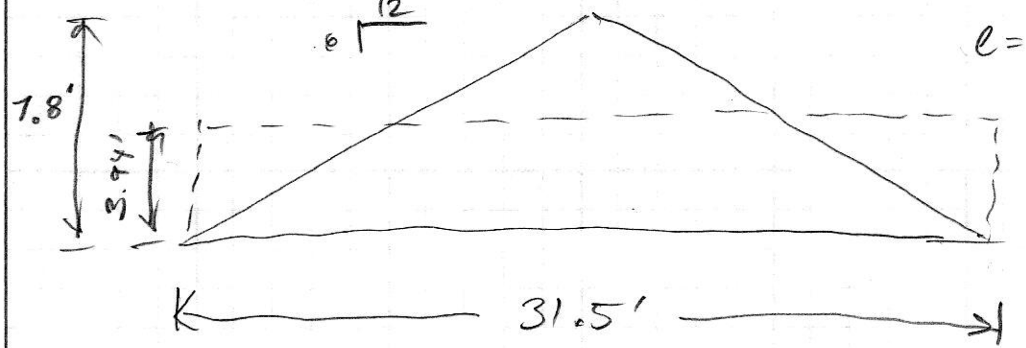
$\ell = 12'' + 16'' = 28''$

$\ell = 2.33'$

Wall Dead Load = 12psf

TCDL+BCDL = 17.8psf

Snow = 19.3psf



$D_{wall} = (12\text{ psf})(3.94') = 47\text{ plf}$

$D_{roof} = 41.5\text{ plf}$

$S = 45\text{ plf}$

$D_{total} = 89\text{ plf}$

2) Garage

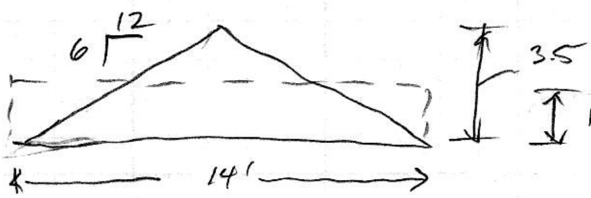
$D_{wall} = (12\text{ psf})(3.25') = 39\text{ plf}$

$D_{roof} = 41.5\text{ plf}$

$S = 45\text{ plf}$

$D_{total} = 81\text{ plf}$

3) Bedroom



$D_{wall} = (12\text{ psf})(1.75') = 21\text{ plf}$

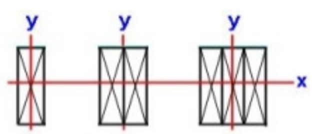
$D_{roof} = 41.5$

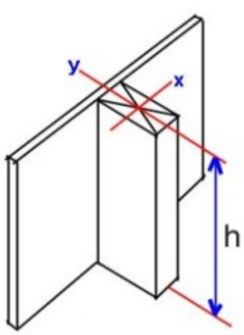
$S = 45\text{ plf}$

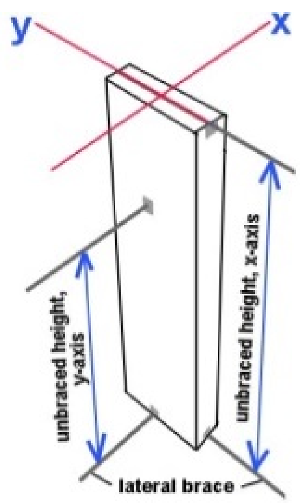
$D_{tot} = 63\text{ plf}$

COLUMNS

Post No.	Type	Grade	Exp.	PT	Bearing	dx	dy	Force (P)	hgt. (H)	lex	ley	ex	ey	Cd	CM_b	CM_c⊥	CM_c	CM_e	Ct	CF_b	CF_c	Ci	Ci_e	Cr	Cb	(le/d)x	(le/d)y	A	Sx	Sy	Kfx	Kfy
P1	(2) 2x6	DF No. 2	dry	N	HF	5.50	3.00	1388	7.0	7.0	1.0	0.92	0.50	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	15.27	4.00	16.50	15.13	8.25	1.00	0.60
P2	4x6	DF No. 2	dry	N	HF	5.50	3.50	2713	8.0	8.0	1.0	0.92	0.58	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	17.45	3.43	19.25	17.65	11.23	1.00	1.00
P3	(2) 2x6	DF No. 2	dry	N	HF	5.50	3.00	4381	8.0	8.0	1.0	0.92	0.50	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	17.45	4.00	16.50	15.13	8.25	1.00	0.60
P4	6x6	DF No. 2	dry	N	HF	5.50	5.50	3102	9.0	9.0	1.0	0.92	0.92	1.15	1	1	1	1	1	1	1	1	1	1	1	19.64	2.18	30.25	27.73	27.73	1.00	1.00
P5	6x6	HF No. 2	wet	Y	DF	5.50	5.50	3615	11.0	11.0	11.0	0.92	0.92	1.15	1	0.67	0.91	1	1	1	1	0.8	0.95	1	1	24.00	24.00	30.25	27.73	27.73	1.00	1.00
P6	(3) 2x6	DF No. 2	dry	N	HF	5.50	4.50	4795	9.0	9.0	1.0	0.92	0.75	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	19.64	2.67	24.75	22.69	18.56	1.00	0.60
P7	4x6	DF No. 2	dry	N	HF	5.50	3.50	508	8.0	8.0	1.0	0.92	0.58	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	17.45	3.43	19.25	17.65	11.23	1.00	1.00
P8	4x6	DF No. 2	dry	N	DF	5.50	3.50	975	10.0	10.0	10.0	0.92	0.58	1.15	1	1	1	1	1	1.3	1.1	1	1	1	1	21.82	34.29	19.25	17.65	11.23	1.00	1.00
P9	6x6	HF No. 2	wet	Y	DF	5.50	5.50	1468	10.0	10.0	10.0	0.92	0.92	1.15	1	0.67	0.91	1	1	1	1	0.8	0.95	1	1	21.82	21.82	30.25	27.73	27.73	1.00	1.00
FP1	4x6	HF No. 2	dry	Y	DF	5.50	3.50	3174	4.0	4.0	4.0	0.92	0.58	1.00	1	1	1	1	1	1.3	1.1	0.8	0.95	1	1	8.73	13.71	19.25	17.65	11.23	1.00	1.00
FP2	4x4	HF No. 2	dry	Y	DF	3.50	3.50	3579	4.0	4.0	4.0	0.58	0.58	1.00	1	1	1	1	1	1.5	1.15	0.8	0.95	1	1	13.71	13.71	12.25	7.15	7.15	1.00	1.00







	2"-4" Thick DF No. 1	2"-4" Thick DF No. 2	2"-4" Thick HF No. 1	2"-4" Thick HF No. 2	
Fc	1500	1350	1350	1300	psi
Fbx	1000	900	975	850	psi
Fby	1000	900	975	850	psi
Ex	1,700,000	1,600,000	1,500,000	1,300,000	psi
Ey	1,700,000	1,600,000	1,500,000	1,300,000	psi
Eminx	620,000	580,000	550,000	470,000	psi
Eminy	620,000	580,000	550,000	470,000	psi

Mech. Lamination Method

Nailed

Post No.	E'minx	E'miny	FcEx	FcEy	Fc*	c	Cpx	Cpy	Cp	Fc'	fc	Axial Check 1	lux	luy	lux/dx	luy/dy	lebx	leby	Rbx	Rby	Fbex	Fbey	Fbe	Fbx*	Fby*
P1	580000	580000	2044	29798	1708	0.80	0.749	0.593	0.593	1012	84	**0.083**	1.00	7.00	2.18	28.00	2.06	12.88	3.89	3.92	46072	45406	45406	1346	1346
P2	580000	580000	1565	40558	1708	0.80	0.660	0.991	0.660	1127	141	**0.125**	1.00	8.00	2.18	27.43	2.06	14.72	3.33	4.52	62710	34055	34055	1346	1346
P3	580000	580000	1565	29798	1708	0.80	0.660	0.593	0.593	1012	266	**0.262**	1.00	8.00	2.18	32.00	2.06	14.72	3.89	4.19	46072	39731	39731	1346	1346
P4	470000	470000	1002	81158	805	0.80	0.761	0.998	0.761	612	103	**0.167**	1.00	9.00	2.18	19.64	2.06	16.56	2.12	6.01	125485	15610	15610	863	863
P5	380000	380000	542	542	481	0.80	0.730	0.730	0.730	352	120	**0.340**	11.00	11.00	24.00	24.00	20.24	20.24	6.65	6.65	10326	10326	10326	529	529
P6	580000	580000	1236	67044	1708	0.80	0.572	0.597	0.572	976	194	**0.198**	1.00	9.00	2.18	24.00	2.06	16.56	2.59	5.44	103663	23544	23544	1346	1346
P7	580000	580000	1565	40558	1708	0.80	0.660	0.991	0.660	1127	26	**0.023**	1.00	8.00	2.18	27.43	2.06	14.72	3.33	4.52	62710	34055	34055	1346	1346
P8	580000	580000	1002	406	1708	0.80	0.491	0.224	0.224	383	51	**0.132**	10.00	10.00	21.82	34.29	18.40	18.40	9.96	5.05	7021	27244	7021	1346	1346
P9	380000	380000	656	656	481	0.80	0.786	0.786	0.786	378	49	**0.128**	10.00	10.00	21.82	21.82	18.40	18.40	6.34	6.34	11359	11359	11359	529	529
FP1	446500	446500	4819	1951	1144	0.80	0.945	0.838	0.838	959	165	**0.172**	4.00	4.00	8.73	13.71	7.90	7.40	6.52	3.20	12596	52184	12596	884	884
FP2	446500	446500	1951	1951	1196	0.80	0.829	0.829	0.829	991	292	**0.295**	4.00	4.00	13.71	13.71	7.40	7.40	5.04	5.04	21133	21133	21133	1020	1020

	5"x5" & Larger DF No. 1	5"x5" & Larger DF No. 2	5"x5" & Larger HF No. 1	5"x5" & Larger HF No. 2	
Fc	1000	700	850	575	psi
Fbx	1200	750	975	575	psi
Fby	1200	750	975	575	psi
Ex	1,600,000	1,300,000	1,300,000	1,100,000	psi
Ey	1,600,000	1,300,000	1,300,000	1,100,000	psi
Eminx	580,000	470,000	470,000	400,000	psi
Eminy	580,000	470,000	470,000	400,000	psi

	PSL 1.7 2650 VERSA-LAM	
Fc	3000	psi
Fbx	2650	psi
Fby	2400	psi
Ex	1,700,000	psi
Ey	1,700,000	psi
Eminx	865,000	psi
Eminy	865,000	psi

Notes:

1. A minimum eccentricity of 1/2 inch or 16.7% of the column width is considered about both principal axes, whichever is greater.

2. The bending moment of the column due to the eccentricities is calculated independently for each axis.

2. Flat use factor in bending interaction equations about any minor axis is conservatively set to unity.

Post No.	CLx	Cly	Fbx'	Fby'	ecc fbx	ecc fby	Bending X Check 2	Bending Y Check 3	Pdelta x	Pdelta y	Amp Ecc x	Amp Ecc y	Combined X Check 4	Combined Y Check 5	Fcperp'	Bearing Check 6
P1	0.999	1.000	1343	1346	84	84	**0.063**	**0.063**	1.04	1.00	1.01	1.00	**0.073**	**0.070**	405.00	**0.208**
P2	0.999	1.000	1344	1346	141	141	**0.105**	**0.105**	1.10	1.00	1.02	1.00	**0.134**	**0.121**	405.00	**0.348**
P3	0.999	1.000	1343	1346	266	266	**0.198**	**0.198**	1.20	1.01	1.04	1.00	**0.317**	**0.269**	405.00	**0.656**
P4	1.000	1.000	863	863	103	103	**0.119**	**0.119**	1.11	1.00	1.02	1.00	**0.164**	**0.147**	405.00	**0.253**
P5	1.000	1.000	529	529	120	120	**0.226**	**0.226**	1.28	1.28	1.05	1.05	**0.421**	**0.421**	418.75	**0.285**
P6	0.999	1.000	1345	1346	194	194	**0.144**	**0.144**	1.19	1.00	1.04	1.00	**0.217**	**0.184**	405.00	**0.478**
P7	0.999	1.000	1344	1346	26	26	**0.020**	**0.020**	1.02	1.00	1.00	1.00	**0.021**	**0.020**	405.00	**0.065**
P8	0.988	1.000	1330	1346	51	51	**0.038**	**0.038**	1.05	1.14	1.01	1.03	**0.058**	**0.062**	625.00	**0.081**
P9	1.000	1.000	529	529	49	49	**0.092**	**0.092**	1.08	1.08	1.02	1.02	**0.117**	**0.117**	418.75	**0.116**
FP1	0.996	1.000	881	884	165	165	**0.188**	**0.187**	1.04	1.09	1.01	1.02	**0.225**	**0.238**	625.00	**0.264**
FP2	1.000	1.000	1020	1020	293	293	**0.287**	**0.287**	1.18	1.18	1.04	1.04	**0.436**	**0.436**	625.00	**0.467**

							Cd =	0.90	Cd =	1.00	Cd =	1.15/1.25	Cd =	1.15/1.25	Cd =	1.60	Cd =	1.60	Cd =	1.60	**Max.**	**Max.**		
Post No.	Dead	Floor Live	Roof Live	Snow	Wind	Seismic	LC1	LC1/Cd	LC2	LC2/Cd	LC3	LC3/Cd	LC4	LC4/Cd	LC5	LC5/Cd	LC6a	LC6a/Cd	LC6b	LC6b/Cd	**Bearing**	**LC**	**Cd**	LC
P1	966	0	0	422	0	0	966	1,073	966	966	1,388	1,207	1,283	1,115	966	604	1,283	802	1,283	802	**1,388**	**1,388**	**1.15**	LC3
P2	1,301	0	0	1,412	0	0	1,301	1,446	1,301	1,301	2,713	2,359	2,360	2,052	1,301	813	2,360	1,475	2,360	1,475	**2,713**	**2,713**	**1.15**	LC3
P3	2,325		0	2,056	0	0	2,325	2,583	2,325	2,325	4,381	3,810	3,867	3,363	2,325	1,453	3,867	2,417	3,867	2,417	**4,381**	**4,381**	**1.15**	LC3
P4	1,387	0	0	1,715	0	0	1,387	1,541	1,387	1,387	3,102	2,697	2,673	2,325	1,387	867	2,673	1,671	2,673	1,671	**3,102**	**3,102**	**1.15**	LC3
P5	1,621	0	0	1,994	0	0	1,621	1,801	1,621	1,621	3,615	3,143	3,117	2,710	1,621	1,013	3,117	1,948	3,117	1,948	**3,615**	**3,615**	**1.15**	LC3
P6	2,492	0	0	2,303	0	0	2,492	2,769	2,492	2,492	4,795	4,170	4,219	3,669	2,492	1,558	4,219	2,637	4,219	2,637	**4,795**	**4,795**	**1.15**	LC3
P7	315	0	0	193	0	0	315	350	315	315	508	442	460	400	315	197	460	287	460	287	**508**	**508**	**1.15**	LC3
P8	400	0	0	575	0	0	400	444	400	400	975	848	831	723	400	250	831	520	831	520	**975**	**975**	**1.15**	LC3
P9	614	0	0	854	0	0	614	682	614	614	1,468	1,277	1,255	1,091	614	384	1,255	784	1,255	784	**1,468**	**1,468**	**1.15**	LC3
FP1	540	2,634	0	0	0	0	540	600	3,174	3,174	540	470	2,516	2,187	540	338	2,516	1,572	2,516	1,572	**3,174**	**3,174**	**1.00**	LC2
FP2	667	2,912	0	0	0	0	667	741	3,579	3,579	667	580	2,851	2,479	667	417	2,851	1,782	2,851	1,782	**3,579**	**3,579**	**1.00**	LC2

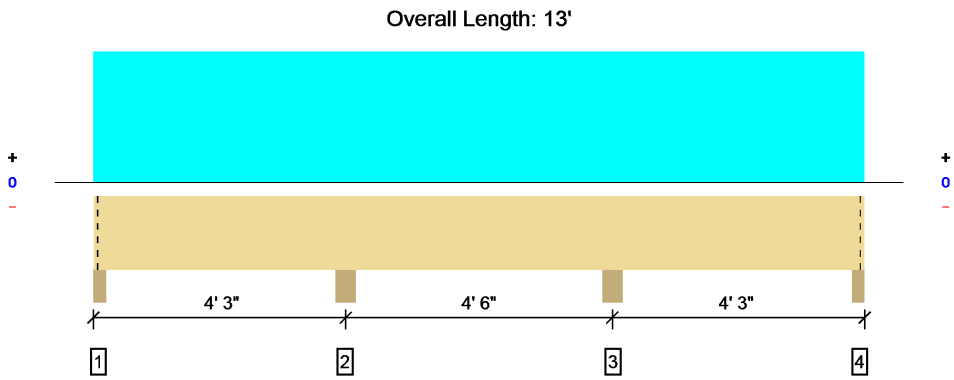
JOB SUMMARY REPORT

2015_048_MEMBERS.4te

01: Level 1			
Member Name	**Results**	**Current Solution**	**Comments**
FJ1	Passed	1 Piece(s) 9 1/2" TJI® 110 @ 16" OC	
FJ2	Passed	1 Piece(s) 9 1/2" TJI® 110 @ 16" OC	
FB1	Passed	1 Piece(s) 4 x 8 Douglas Fir-Larch No. 2	
FB2	Passed	1 Piece(s) 4 x 8 Douglas Fir-Larch No. 2	
FB3	Passed	1 Piece(s) 4 x 8 Douglas Fir-Larch No. 2	
02: Roof			
Member Name	**Results**	**Current Solution**	**Comments**
GT1	Passed	2 Piece(s) 24" TJI® 560D @ 12" OC	Reinforcement accessories required
MB1	Passed	1 Piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
MB2	Passed	1 Piece(s) 6 x 10 Douglas Fir-Larch No. 2	
HD1	Passed	1 Piece(s) 5 1/2" x 16 1/2" 24F-V4 DF Glulam	
HD2	Passed	1 Piece(s) 5 1/2" x 9" 24F-V4 DF Glulam	
HD3	Passed	1 Piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
HD4	Passed	1 Piece(s) 6 x 8 Douglas Fir-Larch No. 2	
HD5	Passed	1 Piece(s) 6 x 10 Douglas Fir-Larch No. 2	
HD6	Passed	1 Piece(s) 6 x 8 Douglas Fir-Larch No. 2	
HD7	Passed	1 Piece(s) 6 x 6 Douglas Fir-Larch No. 2	

FORTE® **MEMBER REPORT** *Level 1, FB3* **PASSED**

1 piece(s) 4 x 8 Douglas Fir-Larch No. 2



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)	3579 @ 8' 9"	12031 (5.50")	Passed (30%)	--	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	1217 @ 3' 5"	3045	Passed (40%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-1465 @ 4' 3"	2989	Passed (49%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.016 @ 2' 1 13/16"	0.136	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.018 @ 2' 1 5/8"	0.204	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Drop Beam
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 13' o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Column - HF	3.50"	3.50"	1.50"	247	1149/-147	1396/-147	Blocking
2 - Column - HF	5.50"	5.50"	1.64"	667	2912	3579	None
3 - Column - HF	5.50"	5.50"	1.64"	667	2912	3579	None
4 - Column - HF	3.50"	3.50"	1.50"	247	1149/-147	1396/-147	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PLF)	0 to 13'	N/A	134.3	570.0	Linked from: FJ2, Support 4

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
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

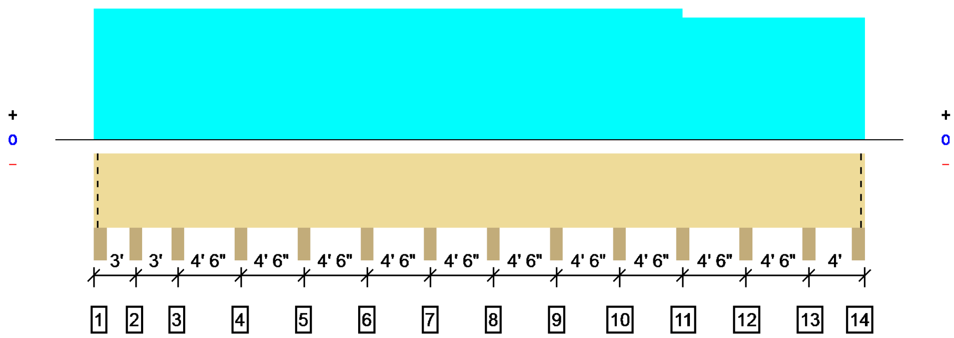
10/31/2015 5:36:40 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

FORTE® **MEMBER REPORT** *Level 1, FB2*

1 piece(s) 4 x 8 Douglas Fir-Larch No. 2

PASSED

Overall Length: 55'



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)	3168 @ 37' 6"	7656 (3.50")	Passed (41%)	--	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	1125 @ 38' 3"	3045	Passed (37%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-1341 @ 37' 6"	2989	Passed (45%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.015 @ 26' 3"	0.150	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.017 @ 39' 9"	0.225	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Drop Beam
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 55' o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Column - HF	3.50"	3.50"	1.50"	158	753/-100	911/-100	Blocking
2 - Column - HF	3.50"	3.50"	1.50"	358	1825	2183	None
3 - Column - HF	3.50"	3.50"	1.50"	457	2248	2705	None
4 - Column - HF	3.50"	3.50"	1.50"	556	2566	3122	None
5 - Column - HF	3.50"	3.50"	1.50"	534	2589	3123	None
6 - Column - HF	3.50"	3.50"	1.50"	540	2622	3162	None
7 - Column - HF	3.50"	3.50"	1.50"	538	2624	3162	None
8 - Column - HF	3.50"	3.50"	1.50"	539	2626	3165	None
9 - Column - HF	3.50"	3.50"	1.50"	536	2625	3161	None
10 - Column - HF	3.50"	3.50"	1.50"	547	2622	3169	None
11 - Column - HF	3.50"	3.50"	1.50"	478	2576	3054	None
12 - Column - HF	3.50"	3.50"	1.50"	406	2495	2901	None
13 - Column - HF	3.50"	3.50"	1.50"	433	2420	2853	None
14 - Column - HF	3.50"	3.50"	1.50"	150	922/-145	1072/-145	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PLF)	0 to 42'	N/A	113.3	493.5	Linked from: FJ1, Support 3
2 - Uniform (PLF)	42' to 55'	N/A	86.3	478.5	Linked from: FJ2, Support 3

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
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

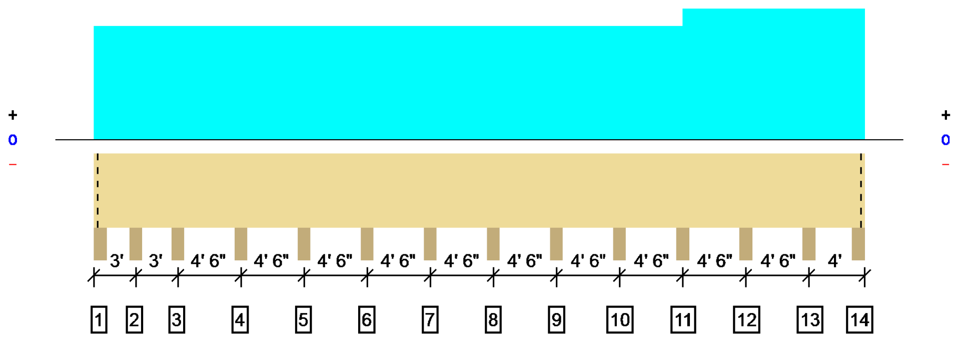
10/31/2015 5:32:34 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

FORTE® **MEMBER REPORT** *Level 1, FB1*

1 piece(s) 4 x 8 Douglas Fir-Larch No. 2

PASSED

Overall Length: 55'



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)	3575 @ 46' 6"	7656 (3.50")	Passed (47%)	--	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	1277 @ 45' 9"	3045	Passed (42%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-1489 @ 46' 6"	2989	Passed (50%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.017 @ 44' 2 15/16"	0.150	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.019 @ 44' 2 15/16"	0.225	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Drop Beam
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 55' o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Column - HF	3.50"	3.50"	1.50"	158	753/-100	911/-100	Blocking
2 - Column - HF	3.50"	3.50"	1.50"	358	1825	2183	None
3 - Column - HF	3.50"	3.50"	1.50"	457	2248	2705	None
4 - Column - HF	3.50"	3.50"	1.50"	556	2566	3122	None
5 - Column - HF	3.50"	3.50"	1.50"	534	2589	3123	None
6 - Column - HF	3.50"	3.50"	1.50"	540	2622	3162	None
7 - Column - HF	3.50"	3.50"	1.50"	538	2625	3163	None
8 - Column - HF	3.50"	3.50"	1.50"	538	2627	3165	None
9 - Column - HF	3.50"	3.50"	1.50"	540	2634	3174	None
10 - Column - HF	3.50"	3.50"	1.50"	532	2631	3163	None
11 - Column - HF	3.50"	3.50"	1.55"	585	2808	3393	None
12 - Column - HF	3.50"	3.50"	1.63"	632	2943	3575	None
13 - Column - HF	3.50"	3.50"	1.60"	649	2854	3503	None
14 - Column - HF	3.50"	3.50"	1.50"	228	1091/-170	1319/-170	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PLF)	0 to 42'	N/A	113.3	493.5	Linked from: FJ1, Support 2
2 - Uniform (PLF)	42' to 55'	N/A	133.5	566.3	Linked from: FJ2, Support 2

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

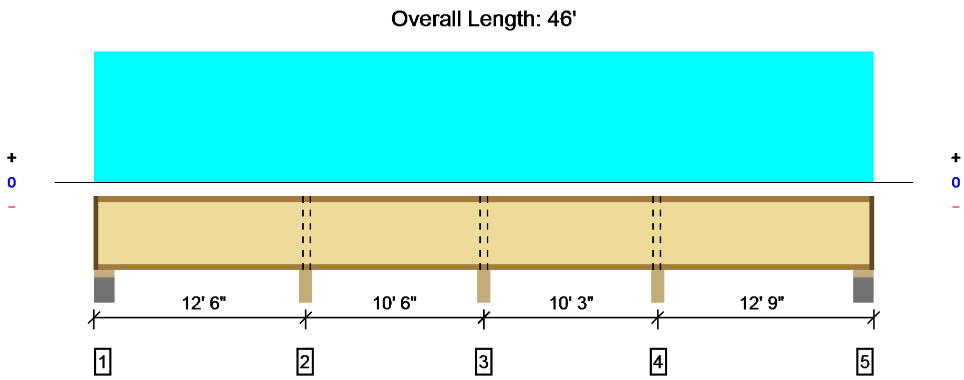
Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

10/31/2015 8:36:06 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

MEMBER REPORT *Level 1, FJ2*

1 piece(s) 9 1/2" TJI® 110 @ 16" OC

PASSED



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)	939 @ 33' 3"	1935 (3.50")	Passed (49%)	1.00	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	469 @ 33' 4 3/4"	1342	Passed (35%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-1077 @ 33' 3"	2500	Passed (43%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.120 @ 39' 9 1/4"	0.309	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.145 @ 39' 9 7/8"	0.412	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
TJ-Pro™ Rating	44	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 4' 2 1/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Plate on concrete - HF	5.50"	4.25"	1.75"	69	303/-19	372/-19	1 1/4" Rim Board
2 - Beam - DF	3.50"	3.50"	3.50"	178	755	933	Blocking
3 - Beam - DF	3.50"	3.50"	3.50"	115	638	753	Blocking
4 - Beam - DF	3.50"	3.50"	3.50"	179	760	939	Blocking
5 - Plate on concrete - HF	5.50"	4.25"	1.75"	71	308/-17	379/-17	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 46'	16"	10.0	40.0	Residential - Living Areas

Member Notes

Bedrooms

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

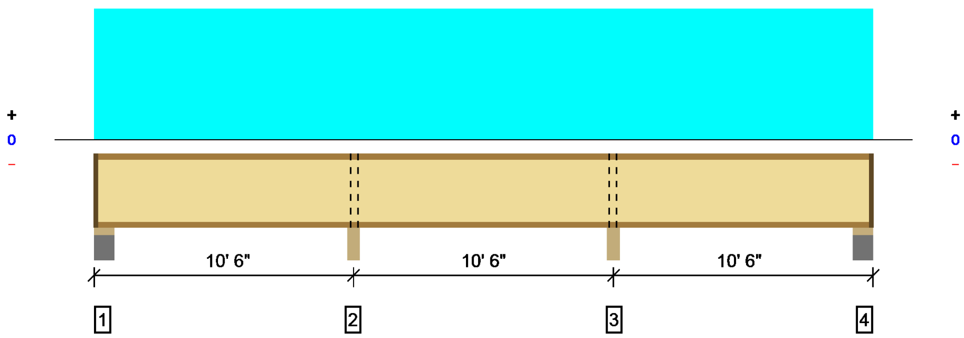
Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

MEMBER REPORT *Level 1, FJ1*

1 piece(s) 9 1/2" TJI® 110 @ 16" OC

PASSED

Overall Length: 31' 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)	809 @ 10' 6"	1935 (3.50")	Passed (42%)	1.00	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	389 @ 21' 1 3/4"	1342	Passed (29%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-798 @ 10' 6"	2500	Passed (32%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.063 @ 5' 2 3/4"	0.253	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.075 @ 26' 3 15/16"	0.338	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
TJ-Pro™ Rating	53	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 4' 10 1/8" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Plate on concrete - HF	5.50"	4.25"	1.75"	59	264/-24	323/-24	1 1/4" Rim Board
2 - Beam - DF	3.50"	3.50"	3.50"	151	658	809	Blocking
3 - Beam - DF	3.50"	3.50"	3.50"	151	658	809	Blocking
4 - Plate on concrete - HF	5.50"	4.25"	1.75"	59	264/-24	323/-24	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 31' 6"	16"	10.0	40.0	Residential - Living Areas

Member Notes

Living Area, Kitchen

Weyerhaeuser Notes

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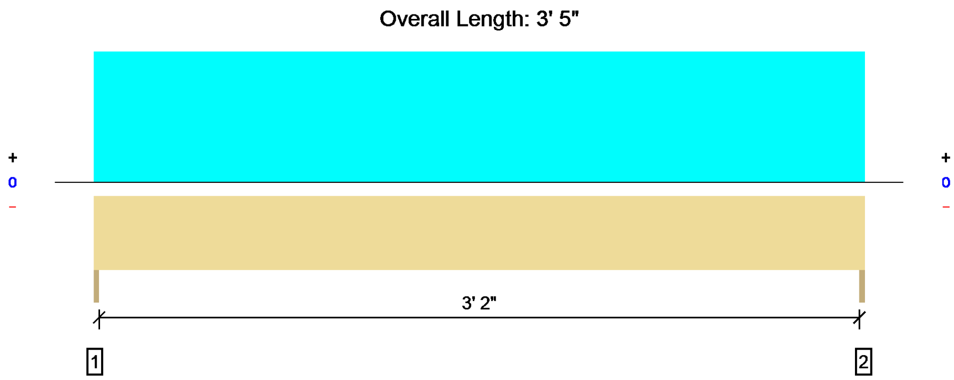
SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

10/31/2015 5:16:55 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

1 piece(s) 6 x 6 Douglas Fir-Larch No. 2



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)	1183 @ 0	5156 (1.50")	Passed (23%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	779 @ 7"	3943	Passed (20%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1011 @ 1' 8 1/2"	1993	Passed (51%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.010 @ 1' 8 1/2"	0.114	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.021 @ 1' 8 1/2"	0.171	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' 5" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	621	562	1183	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	621	562	1183	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 3' 5"	N/A	356.0	329.0	Residential - Living Areas

Member Notes

Garage Entry Door

Weyerhaeuser Notes

SUSTAINABLE FORESTRY INITIATIVE

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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) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

10/31/2015 7:56:56 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

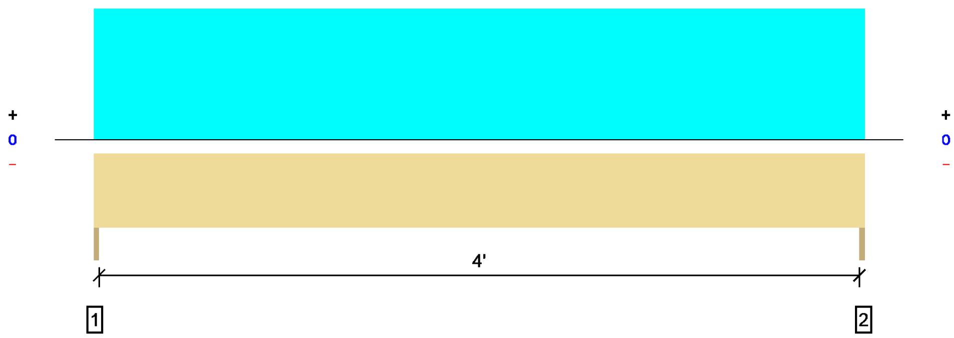
FORTE®

MEMBER REPORT *Roof, HD6*

PASSED

1 piece(s) 6 x 8 Douglas Fir-Larch No. 2

Overall Length: 4' 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)	1136 @ 0	5156 (1.50")	Passed (22%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	735 @ 9"	5376	Passed (14%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1207 @ 2' 1 1/2"	3705	Passed (33%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.008 @ 2' 1 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.016 @ 2' 1 1/2"	0.213	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 4' 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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	536	599	1135	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	536	599	1135	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 4' 3"	N/A	242.0	282.0	Residential - Living Areas

Member Notes

Garage Windows

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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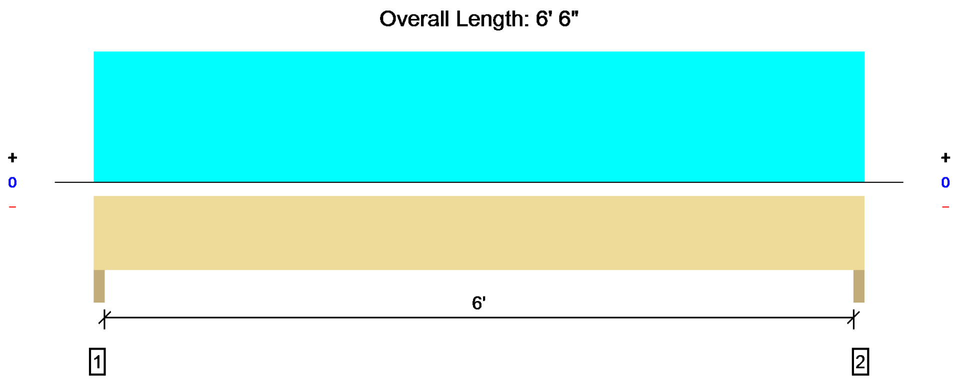
10/31/2015 7:55:12 PM
Forte v5.0, Design Engine: V6.4.0.40
2015_048_MEMBERS.4te

FORTE® **MEMBER REPORT** *Roof, HD5* **PASSED**

1 piece(s) 6 x 10 Douglas Fir-Larch No. 2



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)	2269 @ 1 1/2"	10313 (3.00")	Passed (22%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1542 @ 1' 1/2"	6810	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3409 @ 3' 3"	6937	Passed (49%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.022 @ 3' 3"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.047 @ 3' 3"	0.313	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 6' 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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	1200	1069	2269	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	1200	1069	2269	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 6' 6"	N/A	356.0	329.0	Residential - Living Areas

Member Notes

Entry Door with Sidelights

Weyerhaeuser Notes

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

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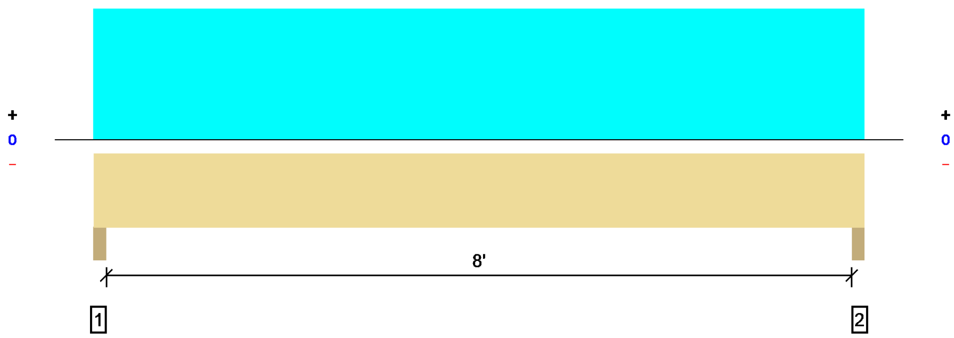
FORTE®

MEMBER REPORT *Roof, HD4*

PASSED

1 piece(s) 6 x 8 Douglas Fir-Larch No. 2

Overall Length: 8' 7"



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)	508 @ 2"	12031 (3.50")	Passed (4%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	400 @ 11"	5376	Passed (7%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1008 @ 4' 3 1/2"	3705	Passed (27%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.019 @ 4' 3 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.049 @ 4' 3 1/2"	0.412	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' 7" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	3.50"	3.50"	1.50"	315	193	508	None
2 - Trimmer - DF	3.50"	3.50"	1.50"	315	193	508	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 8' 7"	N/A	63.0	45.0	Residential - Living Areas

Member Notes

Bay Windows Front

Weyerhaeuser Notes

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

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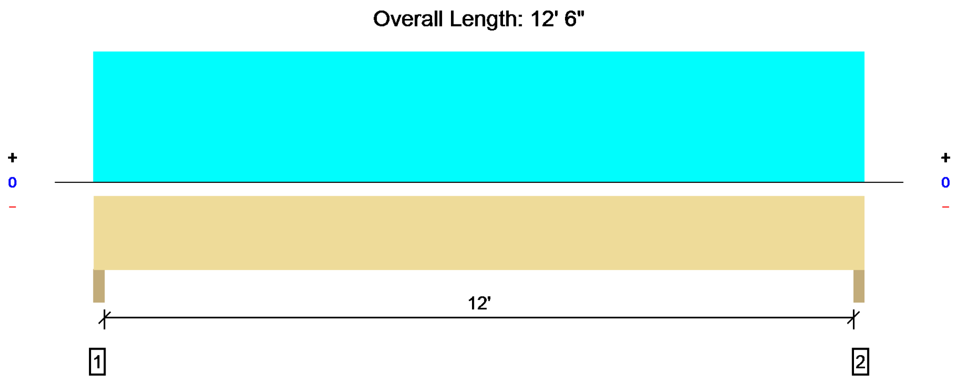
2015_048_MEMBERS.4te

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

1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam



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)	4382 @ 1 1/2"	10725 (3.00")	Passed (41%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3505 @ 1' 3"	13409	Passed (26%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	13150 @ 6' 3"	30360	Passed (43%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.117 @ 6' 3"	0.408	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.249 @ 6' 3"	0.613	Passed (L/590)	--	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 12' 6" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 12' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	2325	2056	4381	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	2325	2056	4381	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 12' 6"	N/A	356.0	329.0	Residential - Living Areas

Member Notes

Patio 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

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10/31/2015 7:43:28 PM
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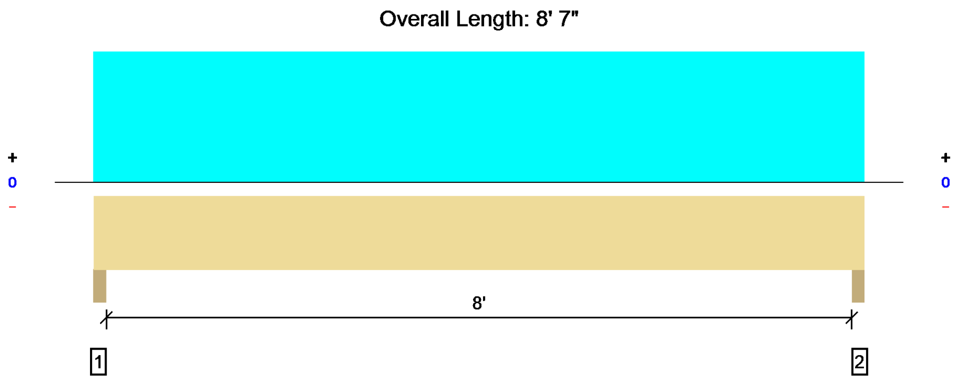
FORTE® **MEMBER REPORT** *Roof, HD2* **PASSED**

1 piece(s) 5 1/2" x 9" 24F-V4 DF Glulam



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)	2712 @ 2"	12513 (3.50")	Passed (22%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2054 @ 1' 1/2"	10057	Passed (20%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	5377 @ 4' 3 1/2"	17078	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.057 @ 4' 3 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.110 @ 4' 3 1/2"	0.412	Passed (L/904)	--	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' 7" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 8' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - DF	3.50"	3.50"	1.50"	1301	1412	2713	None
2 - Trimmer - DF	3.50"	3.50"	1.50"	1301	1412	2713	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 8' 7"	N/A	291.0	329.0	Residential - Living Areas

Member Notes

Bay Windows Rear

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

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10/31/2015 7:34:44 PM
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COMPANY
Medeek Engineering Inc.
3050 State Route 109
Copalis Beach, WA 98535
Nathaniel P. Wilkerson PE
Oct. 31, 2015 19:27

PROJECT
Job#: 2015-048
George Barth
HD1_LATERAL.wwb

Design Check Calculation Sheet

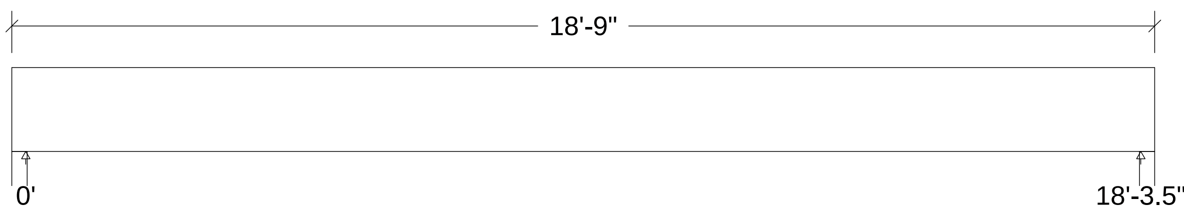
WoodWorks Sizer 10.4

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
CC Wind	Wind	Full Area				48.50(3.00')		psf

Load magnitude does not include Importance factor from Table 4.2.3.2, which is applied during analysis.

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead			
Wind	1364		1364
Factored:			
Total	818		818
Bearing:			
Capacity			
Beam	27720		27720
Support	67319		67319
Anal/Des			
Beam	0.03		0.03
Support	0.01		0.01
Load comb	#2		#2
Length	3.00		3.00
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fc sup	850		850

*Minimum bearing length setting used: 1/2" for end supports

MB1

Glulam-Unbal., West Species, 24F-V4 DF, 5-1/2"x16-1/2"

11 laminations, 5-1/2" maximum width,
Supports: All - Lumber n-ply Column, D.Fir-L Stud
Total length: 18'-9.0"; volume = 11.8 cu.ft.;
Lateral support: top= at supports, bottom= at supports; Oblique angle: 90.0 deg;

Analysis vs. Allowable Stress and Deflection using NDS 2012 :

Criterion		Analysis Value	Design Value	Unit	Analysis/Design (%)
Shear	x-x	fv = 0	Fv' = 424	kips	fv/Fv' = 0.0
	y-y	fv = 13	Fv' = 368	psi	fv/Fv' = 3.4
Bending(+)	x-x	fb = 0	Fb' = 3373	kip-ft	fb/Fb' = 0.0
	y-y	fb = 527	Fb' = 2528	kip-ft	fb/Fb' = 20.8
Live Defl'n		0.60 = L/365	0.61 = L/360	in	98.5
Total Defl'n		0.60 = L/365	0.91 = L/240	in	65.7

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fvy'	230	1.60	1.00	1.00	-	-	-	-	1.00	1.00	-	2
Fby'	1450	1.60	1.00	1.00	0.999	1.000	1.09	1.00	1.00	1.00	-	2
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
Ey'	1.6 million		1.00	1.00	-	-	-	-	1.00	-	-	2
Emin'	0.95 million		1.00	1.00	-	-	-	-	1.00	-	-	2

Only the lesser of CL and CV is applied, as per NDS 5.3.6

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = .6D+.6W, V =    798, V design =    757 lbs
 Bending(+): LC #2  = .6D+.6W, M =   3651 lbs-ft
 Deflection: LC #2  = .6D+.6W  (live)
             LC #2  = .6D+.6W  (total)
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-10 / IBC 2012
```

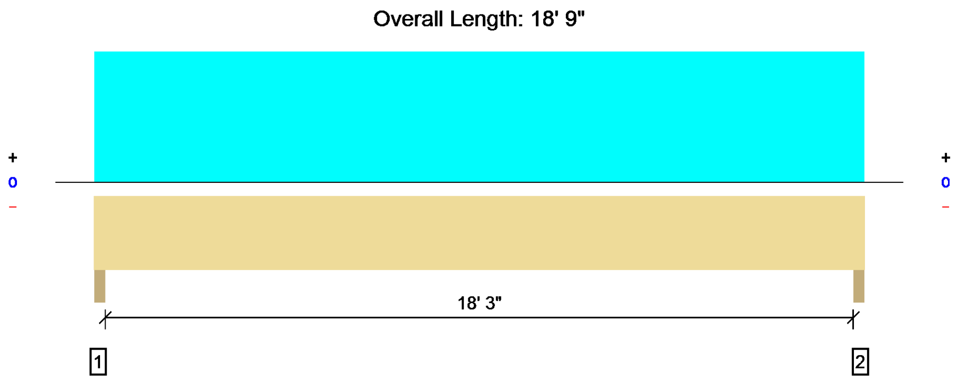
CALCULATIONS:

```
 Deflection:  EI =   3706e06 lb-in2     EIy =    366e06 lb-in2
 "Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
 Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
 Lateral stability (+): Lu = 18'-3.50"  Le = 33'-7.88"  RB =    2.86
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2012), the National Design Specification (NDS 2012), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2010 and manufactured in accordance with ANSI A190.1-2007
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).
7. Design by: Nathaniel P. Wilkerson PE

1 piece(s) 5 1/2" x 16 1/2" 24F-V4 DF Glulam



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)	1388 @ 1 1/2"	10725 (3.00")	Passed (13%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1147 @ 1' 7 1/2"	18437	Passed (6%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	6334 @ 9' 4 1/2"	55913	Passed (11%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.032 @ 9' 4 1/2"	0.617	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.105 @ 9' 4 1/2"	0.925	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 18' 9" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Critical positive moment adjusted by a volume factor of 0.97 that was calculated using length L = 18' 6".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Total	Accessories
1 - Trimmer - DF	3.00"	3.00"	1.50"	966	422	1388	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	966	422	1388	None

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 18' 9"	N/A	81.0	45.0	Residential - Living Areas

Member Notes

Garage Door

Weyerhaeuser Notes

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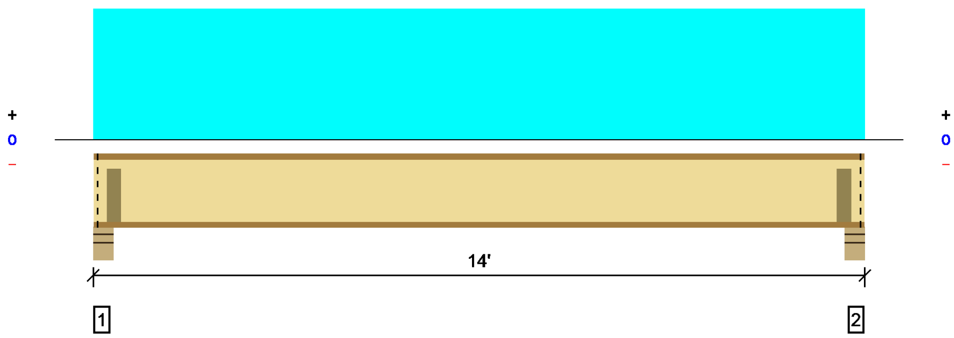
SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Nathaniel Wilkerson Medeek Engineering Inc. (425) 420-5715 nathan@medeek.com	Job#: 2015-048 George Barth Ocean Shores, WA 98569

FORTE® **MEMBER REPORT** *Roof, GT1* **PASSED**

2 piece(s) 24" TJI® 560D @ 12" OC

Overall Length: 14'



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)	4795 @ 4 1/2"	6509 (3.50")	Passed (74%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	4481 @ 5 1/2"	7820	Passed (57%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	15033 @ 7'	45310	Passed (33%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.064 @ 7'	0.442	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.132 @ 7'	0.663	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC
Design Methodology : ASD
Member Pitch: 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 8' 4 9/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Total	Accessories
1 - Stud wall - DF	5.50"	5.50"	1.75"	2492	2303	4795	Web Stiffeners, Blocking
2 - Stud wall - DF	5.50"	5.50"	1.75"	2492	2303	4795	Web Stiffeners, Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 14'	N/A	356.0	329.0	Roof

Member Notes

Girder Truss @ Bedroom

Weyerhaeuser Notes

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

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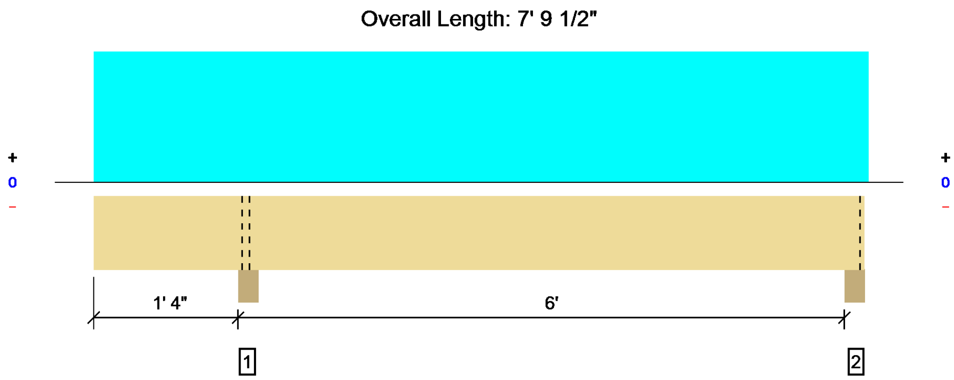
FORTE® **MEMBER REPORT** *Roof, MB2* **PASSED**

1 piece(s) 6 x 10 Douglas Fir-Larch No. 2



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)	1468 @ 1' 6 3/4"	18906 (5.50")	Passed (8%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	664 @ 2' 7"	6810	Passed (10%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1221 @ 4' 7 7/8"	6937	Passed (18%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.009 @ 4' 6 5/8"	0.197	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.015 @ 4' 6 13/16"	0.295	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 7' 9 1/2" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Column - HF	5.50"	5.50"	1.50"	614	854	1468	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	400	575	975	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 7' 9 1/2"	N/A	117.0	181.0	Residential - Living Areas

Member Notes

Deck Roof

Weyerhaeuser Notes

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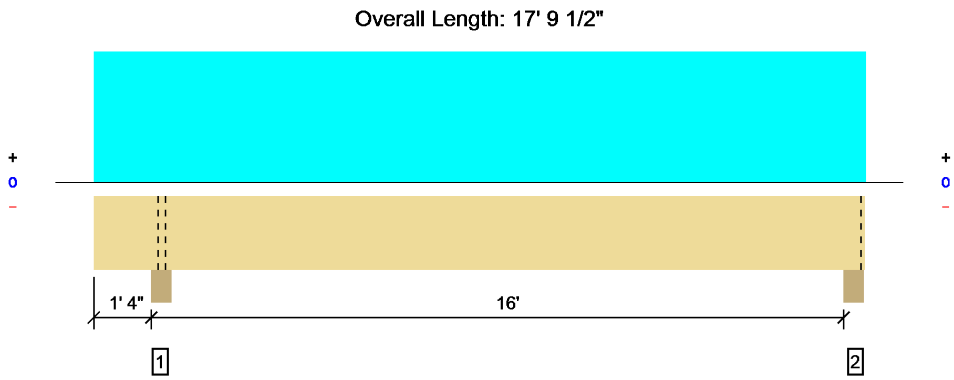
FORTE® **MEMBER REPORT** *Roof, MB1* **PASSED**

1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam



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)	3615 @ 1' 6 3/4"	19663 (5.50")	Passed (18%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2562 @ 2' 9 1/2"	13409	Passed (19%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	11743 @ 9' 6 13/16"	30360	Passed (39%)	1.15	1.0 D + 1.0 S (Alt Spans)
Neg Moment (Ft-lbs)	-460 @ 1' 6 3/4"	23403	Passed (2%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.207 @ 9' 6 1/4"	0.530	Passed (L/921)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.374 @ 9' 6 5/16"	0.795	Passed (L/511)	--	1.0 D + 1.0 S (Alt Spans)

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 17' 9 1/2" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 15' 9 7/16".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 1' 8 9/16".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Column - HF	5.50"	5.50"	1.50"	1621	1994	3615	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	1387	1715	3102	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PLF)	0 to 17' 9 1/2"	N/A	153.0	208.0	Residential - Living Areas

Member Notes

Deck Roof

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TRUSS UPLIFT CALCULATIONS

Out-to-out Span	31.5	ft	Load Combo:	.6D + .6W
Overhang Left	1.33	ft		
Overhang Right	0	ft		
Truss O/C Spacing	2	ft		
Roof Pitch	6	:12	26.57	Deg.

Dead Loads

BCDL	10	psf	SF (Slope Factor) =	1.12	
TCDL	7	psf	Adj. TCDL =	7.83	psf

Wind Loads (MWFRS)

Wind (0 to h/2)	42.08	psf	(Wind loads are strength design pressures)
Wind (0 to h/2 @ overhang)	34.07	psf	
Wind (overhang underside)	30.18	psf	

Moment Summation at Truss Left Bearing:

	Force		Moment-Arm		Moment	
BCDL	-378.0	lbs	15.8	ft	-5953.5	ft-lbs
TCDL	-295.8	lbs	15.8	ft	-4659.4	ft-lbs
TCDL (right overhang)	0.0	lbs	31.5	ft	0.0	ft-lbs
TCDL (left overhang)	-12.5	lbs	-0.7	ft	8.3	ft-lbs
Wind (0 to h/2)	1590.6	lbs	15.8	ft	25052.3	ft-lbs
Wind (right overhang)	0.0	lbs	31.5	ft	0.0	ft-lbs
Wind (left overhang)	102.5	lbs	-0.7	ft	-68.2	ft-lbs
Totals	1006.8	lbs	108.9		14379.6	ft-lbs

Uplift Right	**456.5**	**lbs**
Uplift Left	**550.3**	**lbs**

Use H1 Hurricane Ties for all rafters and trusses.
Use LGT3 or LGT2 at Girder 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).

Stemwall Cont. Footing Calculator

Check continuous footings at highest (vertically) loaded section of wall excluding point loads.
From previous sections and by inspection the most critically loaded wall is at Wall Line 1.

(plf)	Dead Load	Floor Live	Roof Live	0.6(Wind)
Roof	356	0	330	0
Wall	120	0	0	0
Floor	53	210	0	0
Stemwall	150	0	0	0
Wind	0	0	0	194
Totals	679	210	330	194

Roof LL or S =	19.3	psf
Roof DL =	20.9	psf
Roof Trib. Width =	17.1	ft
Wall DL =	12	psf
Wall Hgt. =	10	ft
Floor LL =	40.0	psf
Floor DL =	10.0	psf
FloorTrib. Width =	5.3	ft
Wind (ASD) =	194.0	plf
ρ_{conc} =	150	pcf
Steel Yield Strength =	60,000	psi
Conc. Comp. Strength =	3,000	psi
Soil Bearing Pressure =	1,500	psf
Reinf. Cover =	3	in
Reinf. Bar Size =	4	
Soil Depth Above Ftg.	12	in
ρ_{soil} =	100	pcf
Stem Width =	6	in
Stem Hgt. =	24	in
Footing Width =	16	in
Footing Depth =	8	in

ASD Load Cases from ASCE 7-10:

2.) D + L =	889	plf	
3.) D + (Lr or S) =	1,008	plf	
6a.) D + .75L + .75(.6W) + .75(Lr or S) =	1,229	plf	(governs)

<u>Bearing Calculations:</u>

Applied Bearing Pressure	Qasd =	922	psf	
Eff. Allowable SBP	Qe =	1,338	psf	
Footing Width Required	Wreq =	11.0	in	
Footing Width	$W_{footing}$ =	**16**	in	⟶ **OK**

Strength Design Load Cases from ASCE 7-10:

1.) 1.4D =	950	plf	
2.) 1.2D + 1.6L + .5(Lr or S) =	1,315	plf	
3.) 1.2D + 1.6(Lr or S) + L =	1,552	plf	(governs)
4.) 1.2D + 1.0W + L + .5(Lr or S) =	1,512	plf	

<u>Beam Shear Calculations (One Way Shear):</u>

Ult. Applied Bearing Pressure	Qu =	1,164	psf	
Applied Beam Shear	Vu =	73	lbs	
Allowable Beam Shear	Vc =	4,190	lbs (ACI 11-3)	
Footing Depth Required	Dreq =	0.1	in	
Footing Depth	$D_{footing}$ =	**8.0**	in	⟶ **OK**

<u>Beam Shear Calculations (One Way Shear):</u>
<u>Unreinforced Concrete</u>

Vu =	485	lbs	
Vc =	3,155	lbs (ACI 22-9)	
Dreq =	1.2	in	
$D_{footing}$ =	**8.0**	in	⟶ **OK**

<u>Bending Calculations:</u>

	a =	0.38	in
Cantilever length	Lcant =	5.0	in
Factored Bending Moment	Mu =	1,212	in-lb
Moment Strength	Mn =	43,021	in-lb

<u>Bending Calculations:</u>
<u>Unreinforced Concrete</u>

S =	72.0	in^3	
Mu =	1,212	in-lb	
Mn =	11,831	in-lb (ACI 22-2)	
Dreq =	0.8	in	
$D_{footing}$ =	**8.0**	in	⟶ **OK**

<u>Transverse Reinforcement Calculations:</u>

$Mu/\phi bd^2$	Rn =	6.2	psi	
Steel Ratio	**ρ =**	0.0001		
Steel Req. based on Moment	As(1) =	0.005	in^2	
Steel Req. based on Shrink	As(2) =	0.173	in^2 (ACI 7.12)	
Controlling Reinf. Steel	As(req) =	0.173	in^2	
Required Spacing with #4 bars =		13.64	in o/c	
Selected Transverse Spacing:	**#4 bars @**	**12**	**in o/c**	
Reinforcement Area Provided	As =	0.196	in^2	⟶ **OK**

<u>Eff. Depth to Top Layer of Steel</u>

d = 4.25 in

(Transverse Reinforcment Unnecessary)

<u>Development Length Calculations:</u>

Note: Plain concrete adequate for bending, therefore development length not required.

spacing/cover dimension	c =	3.0	in
Transverse Reinf. Factor	$c + K_{tr}/d_b$ =	6	(use 2.5)
Length Req.	Ld =	11.6	in (ACI 12-1)
Length Available	Ld-sup =	2	in

λ =	1.0	(lightweight aggregate factor)
ψ_t =	1.0	(reinforcement location factor)
ψ_e =	1.0	(coating factor)
ψ_s =	0.8	(reinforcement size factor)
K_{tr} =	0.0	(transverse reinf. Index)

<u>Longitudinal Reinforcement Calculations:</u>

Steel Req. based on Shrink	As(2) =	0.230	in^2 (ACI 7.12)	
Controlling Reinf. Steel	As(req) =	0.230	in^2	
Required number of #4 bars =		1.17		
Selected Longitudinal Bars:		**2**	**- Cont. #4 bars**	
Reinforcement Area Provided	As =	0.393	in^2	⟶ **OK**

Square Footing Calculator **Footing at P9**

Check square pad footing at location of column.

By inspection the dead and live loads acting vertically on this column are:

(lbs)	Dead Load	Floor Live	Roof Live
Roof	614	0	854
Floor	0	0	0
Totals	614	0	854

Roof LL or S =	854.0	lbs
Roof DL =	614.0	lbs
Floor LL =	0.0	lbs
Floor DL =	0.0	lbs
Column Width =	5.50	in
Column Breadth =	5.50	in
Column Type =	WOOD	
ρ_{conc} =	150	pcf
Steel Yield Strength =	60,000	psi
Conc. Comp. Strength =	3,000	psi
Soil Bearing Pressure =	1,500	psf
Reinf. Cover =	3	in
Reinf. Bar Size =	4	
Soil Depth Above Ftg.	24	in
ρ_{soil} =	100	pcf
Footing Width =	18	in
Footing Depth =	12	in
Equivalent Footing Dia. =	20.31	in

ASD Load Cases from ASCE 7-10:

2.) D + L =	614	lbs	
3.) D + (Lr or S) =	1468	lbs	(governs)
4.) D + .75L + .75(Lr or S) =	1254.5	lbs	

Bearing Calculations:

Applied Bearing Pressure	Qasd =	652	psf	
Eff. Allowable SBP	Qe =	1,150	psf	
Footing Area Required	Areq =	1.28	ft^2	
Area of Footing	Afooting =	**2.25**	ft^2	⟶ **OK**
Weight to resist Uplift w/ 1.5 F.S.	U.R. =	497	lbs	

Strength Design Load Cases from ASCE 7-10:

1.) 1.4D =	859.6	lbs	
2.) 1.2D + 1.6L + .5(Lr or S) =	1163.8	lbs	
3.) 1.2D + 1.6(Lr or S) + L =	2103.2	lbs	(governs)

Eff. Depth to Top Layer of Steel

d = 8.250 in

Beam Shear Calculations (One Way Shear):

Ult. Applied Bearing Pressure	Qu =	935	psf	
Applied Beam Shear	Vu1 =	-234	lbs	
Allowable Beam Shear	Vc1 =	12,201	lbs (ACI 11-3)	
Footing Depth Required	Dreq =	-0.2	in	
Footing Depth	Dfooting =	**12.0**	in	⟶ **OK**

Baseplate Bearing Calculations:

$\sqrt{A_2/A_1}$ =	3.27		
Pu =	2,103	lbs	
Pallow =	100,279	lbs (ACI 10.14)	
Areq =	0.6	in^2	
A1 =	**30.3**	in^2	⟶ **OK**

Punching Shear Calculations (Two Way Shear):

Critical Perimeter	b0 =	55.0	in	
Column Ratio	βc =	1.0		
Column Location Factor	αs =	20		
Punching Shear	Vu2 =	876	lbs	
Allowable Punching Shear	Vc2-a =	111,838	lbs (ACI 11-31)	
Allowable Punching Shear	Vc2-b =	93,198	lbs (ACI 11-32)	
Allowable Punching Shear	Vc2-c =	74,559	lbs (ACI 11-33)	
Controlling Punching Shear	Vc2 =	74,559	lbs	
Footing Depth Required	Dreq =	0.1	in	
Footing Depth	Dfooting =	**12.0**	in	⟶ **OK**

Bending Calculations:

Cantilever length	Lcant =	6.25	in
Conc. Comp. Block	a =	0.51	in
Bending Moment	Mu =	2,282	in-lb
Moment Strength	Mn =	169,505	in-lb

Bending Calculations:
Unreinforced Concrete

S =	300.0	in^3	
Mu =	2,282	in-lb	
Mn =	49,295	in-lb (ACI 22-2)	
Dreq =	0.6	in	
Dfooting =	**12.0**	in	⟶ **OK**

Reinforcement Calculations:

$M_u/\phi b d^2$	Rn =	2.1	psi	
Steel Ratio	ρ =	0.0000		
Steel Req. based on Moment	As(1) =	0.005	in^2	
Steel Req. based on Shrink	As(2) =	0.389	in^2 (ACI 7.12)	
Controlling Reinf. Steel	As(req) =	0.389	in^2	
Required number of # bars =		1.98		
Selected Longitudinal Bars:		**2**	**- #4 bars each way**	
Reinforcement Area Provided	As =	0.39	in^2	⟶ **OK**

Development Length Calculations:

spacing/cover dimension	c =	3.0	in
Transverse Reinf. Factor	$c + K_{tr}/d_b$ =	6	(use 2.5)
Length Req.	Ld =	13.0	in (ACI 12-1)
Length Available	Ld-sup =	3.25	in

λ =	1.0	(lightweight aggregate factor)
ψ_t =	1.0	(reinforcement location factor)
ψ_e =	1.0	(coating factor)
ψ_s =	0.8	(reinforcement size factor)
K_{tr} =	0.0	(transverse reinf. Index)

Note: Plain concrete adequate for bending, therefore development length not required.

Square Footing Calculator **Footing at P5**

Check square pad footing at location of column.

By inspection the dead and live loads acting vertically on this column are:

(lbs)	Dead Load	Floor Live	Roof Live
Roof	1621	0	1994
Floor	0	0	0
Totals	1621	0	1994

Roof LL or S =	1994.0	lbs
Roof DL =	1621.0	lbs
Floor LL =	0.0	lbs
Floor DL =	0.0	lbs
Column Width =	5.50	in
Column Breadth =	5.50	in
Column Type =	WOOD	
ρ_{conc} =	150	pcf
Steel Yield Strength =	60,000	psi
Conc. Comp. Strength =	3,000	psi
Soil Bearing Pressure =	1,500	psf
Reinf. Cover =	3	in
Reinf. Bar Size =	4	
Soil Depth Above Ftg.	12	in
ρ_{soil} =	100	pcf
Footing Width =	24	in
Footing Depth =	12	in
Equivalent Footing Dia. =	27.08	in

ASD Load Cases from ASCE 7-10:

2.) D + L =	1621	lbs	
3.) D + (Lr or S) =	3615	lbs	(governs)
4.) D + .75L + .75(Lr or S) =	3116.5	lbs	

Bearing Calculations:

Applied Bearing Pressure	Qasd =	904	psf	
Eff. Allowable SBP	Qe =	1,250	psf	
Footing Area Required	Areq =	2.89	ft^2	
Area of Footing	Afooting =	**4.00**	ft^2	→ **OK**
Weight to resist Uplift w/ 1.5 F.S.	U.R. =	653	lbs	

Strength Design Load Cases from ASCE 7-10:

1.) 1.4D =	2269.4	lbs	
2.) 1.2D + 1.6L + .5(Lr or S) =	2942.2	lbs	
3.) 1.2D + 1.6(Lr or S) + L =	5135.6	lbs	(governs)

Eff. Depth to Top Layer of Steel

d = 8.250 in

Beam Shear Calculations (One Way Shear):

Ult. Applied Bearing Pressure	Qu =	1,284	psf	
Applied Beam Shear	Vu1 =	214	lbs	
Allowable Beam Shear	Vc1 =	16,267	lbs (ACI 11-3)	
Footing Depth Required	Dreq =	0.2	in	
Footing Depth	Dfooting =	**12.0**	in	→ **OK**

Baseplate Bearing Calculations:

$\sqrt{A_2/A_1}$ =	4.36		
Pu =	5,136	lbs	
Pallow =	100,279	lbs (ACI 10.14)	
Areq =	1.5	in^2	
A1 =	**30.3**	in^2	→ **OK**

Bending Calculations:

Cantilever length	Lcant =	9.25	in
Conc. Comp. Block	a =	0.58	in
Bending Moment	Mu =	9,154	in-lb
Moment Strength	Mn =	253,236	in-lb

Punching Shear Calculations (Two Way Shear):

Critical Perimeter	b0 =	55.0	in	
Column Ratio	βc =	1.0		
Column Location Factor	αs =	20		
Punching Shear	Vu2 =	3,450	lbs	
Allowable Punching Shear	Vc2-a =	111,838	lbs (ACI 11-31)	
Allowable Punching Shear	Vc2-b =	93,198	lbs (ACI 11-32)	
Allowable Punching Shear	Vc2-c =	74,559	lbs (ACI 11-33)	
Controlling Punching Shear	Vc2 =	74,559	lbs	
Footing Depth Required	Dreq =	0.6	in	
Footing Depth	Dfooting =	**12.0**	in	→ **OK**

Bending Calculations:
Unreinforced Concrete

S =	400.0	in^3	
Mu =	9,154	in-lb	
Mn =	65,727	in-lb (ACI 22-2)	
Dreq =	1.7	in	
Dfooting =	**12.0**	in	→ **OK**

Reinforcement Calculations:

Mu/ϕbd^2	Rn =	6.2	psi	
Steel Ratio	ρ =	0.0001		
Steel Req. based on Moment	As(1) =	0.021	in^2	
Steel Req. based on Shrink	As(2) =	0.518	in^2 (ACI 7.12)	
Controlling Reinf. Steel	As(req) =	0.518	in^2	
Required number of # bars =		2.64		
Selected Longitudinal Bars:		**3**	**- #4 bars each way**	
Reinforcement Area Provided	As =	0.59	in^2	→ **OK**

Development Length Calculations:

spacing/cover dimension	c =	3.0	in
Transverse Reinf. Factor	c + K_{tr}/d_b =	6	(use 2.5)
Length Req.	Ld =	11.6	in (ACI 12-1)
Length Available	Ld-sup =	6.25	in

λ =	1.0	(lightweight aggregate factor)
ψ_t =	1.0	(reinforcement location factor)
ψ_e =	1.0	(coating factor)
ψ_s =	0.8	(reinforcement size factor)
K_{tr} =	0.0	(transverse reinf. Index)

Note: Plain concrete adequate for bending, therefore development length not required.

Job#:	**2015-048**
Location:	Wall Line 1

STUD WALL CALCULATIONS

Stud Width (dy)	1.50	in
Stud Depth (dx)	5.50	in
Stud Length (L)	9.00	ft
Stud Spacing	16	in
Stud Species and Grade	2X6 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	1.00	
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	

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	19.64	(governs)
E'min	510,000	psi
FcE	1087.23	psi
Fc*	977.50	psi
c	0.80	sawn lumber
FcE/Fc*	1.112	
1 + FcE/Fc*/2c	1.320	
Cp	0.726	
Fc'	710.09	psi
fc	110.71	psi
CSI (axial)	**0.16**	**OK**

Bearing on Stud Wall Plates

lb	1.50	in
Cb	1.00	(conservative)
Fc⊥'	405.00	psi
fc⊥	110.71	psi
CSI (bearing)	**0.27**	**OK**

Deflection

E'	1,400,000	psi
ΔWIND (.42C&C)*	0.18	in
L/d**	**598**	**OK**

*IBC 2015 Sec. 1604.3

**IRC 2015 Sec. 301.7

Vertical Loads

Wall LL (wLL)	329	plf
Wall DL (wDL)	356	plf
Wall DL (wTL)	685	plf
Trib. Length	1.33	ft
Pc	913.33	lbs

Lateral Loads (Wind MWFRS)

Wind Load (windward wall)	38.19	psf
MWFRS Wind Load ASD	22.91	psf
Wind Atrib	12.00	ft^2
W	274.97	lbs
w	30.55	plf

Lateral Loads (Wind C&C)

Wind Load (Zone 5)	63.58	psf
CC Wind Load ASD	38.15	psf
W	457.78	lbs
w	50.86	plf

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

Mmax	515.00	ft-lbs
	6179.98	in-lbs
fbx	817.19	psi
CSI (bending C&C)	**0.64**	**OK**

Load Case 3: Gravity Loads and Lateral Loads

CD	1.60	(Wind/Seismic)
Mmax	309.34	ft-lbs
	3712.07	in-lbs
CL	0.99	
Cr	1.15	@ 16 O/C
Fbx'	1278.76	psi
fbx	490.85	psi
CSI (bending MWFRFS)	**0.38**	**OK**

Combined Stress

(re-evaluate compression values with CD = 1.6)

FcEx	1087.23	psi
FcE	1087.23	psi
Fc*	1360.00	psi
c	0.80	sawn lumber
FcE/Fc*	0.799	
1 + FcE/Fc*/2c	1.125	
Cp	0.609	
Fc'	828.71	psi

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}'}\right) = \mathbf{0.45}\ \textbf{OK}$$

Load Case: LCMAX

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

Location:	All 9' high walls of residence.
Specification:	**Use 2X6 DF Stud Grade @ 16" o/c**

			Job#:	2015-048

STUD WALL CALCULATIONS — Location: Wall Line A

Stud Width (dy)	1.50	in	**Vertical Loads**		
Stud Depth (dx)	5.50	in	Wall LL (wLL)	282	plf
Stud Length (L)	10.00	ft	Wall DL (wDL)	242	plf
Stud Spacing	16	in	Wall DL (wTL)	524	plf
Stud Species and Grade	2X6 DF Stud		Trib. Length	1.33	ft
Top/Sill Plt. Species	HF		Pc	698.67	lbs

Design Values			**Lateral Loads (Wind MWFRS)**		
Fb	700	psi	Wind Load (windward wall)	38.19	psf
Fc	850	psi	MWFRS Wind Load ASD	22.91	psf
Fc⊥	405	psi	Wind Atrib	13.33	ft^2
E	1,400,000	psi	W	305.52	lbs
Emin	510,000	psi	w	30.55	plf
CF_b	1.00				
CF_c	1.00		**Lateral Loads (Wind C&C)**		
A	8.25	in^2	Wind Load (Zone 5)	63.58	psf
Sx	7.56	in^3	CC Wind Load ASD	38.15	psf
Ix	20.80	in^4	W	508.64	lbs
Ct_c	1.00		w	50.86	plf
CM_c	1.00				
Ci_c	1.00		**Load Case 2: Lateral Loads Only (Wind C&C)**		
			Mmax	635.80	ft-lbs
Load Case 1: Gravity Loads Only				7629.60	in-lbs
ly (unbraced length)	1.0	ft	fbx	1008.87	psi
CD	1.15	(Snow Load)	**CSI (bending C&C)**	**0.79**	**OK**
(le/d)y	8.00				
(le/d)x	21.82	(governs)	**Load Case 3: Gravity Loads and Lateral Loads**		
E'min	510,000	psi	CD	1.60	(Wind/Seismic)
FcE	880.65	psi	Mmax	381.90	ft-lbs
Fc*	977.50	psi		4582.80	in-lbs
c	0.80	sawn lumber	CL	0.99	
FcE/Fc*	0.901		Cr	1.15	@ 16 O/C
1 + FcE/Fc*/2c	1.188		Fbx'	1278.76	psi
Cp	0.654		fbx	605.99	psi
Fc'	639.16	psi	**CSI (bending MWFRFS)**	**0.47**	**OK**
fc	84.69	psi			
CSI (axial)	**0.13**	**OK**	**Combined Stress**		
			(re-evaluate compression values with CD = 1.6)		
Bearing on Stud Wall Plates			FcEx	880.65	psi
lb	1.50	in	FcE	880.65	psi
Cb	1.00	(conservative)	Fc*	1360.00	psi
Fc⊥'	405.00	psi	c	0.80	sawn lumber
fc⊥	84.69	psi	FcE/Fc*	0.648	
CSI (bearing)	**0.21**	**OK**	1 + FcE/Fc*/2c	1.030	
			Cp	0.529	
Deflection			Fc'	719.21	psi
E'	1,400,000	psi			
ΔWIND (.42C&C)*	0.28	in			
L/d**	**436**	**OK**			

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}'}\right) = \mathbf{0.54}\ \textbf{OK}$$

*IBC 2015 Sec. 1604.3

**IRC 2015 Sec. 301.7

Load Case: LCMAX

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

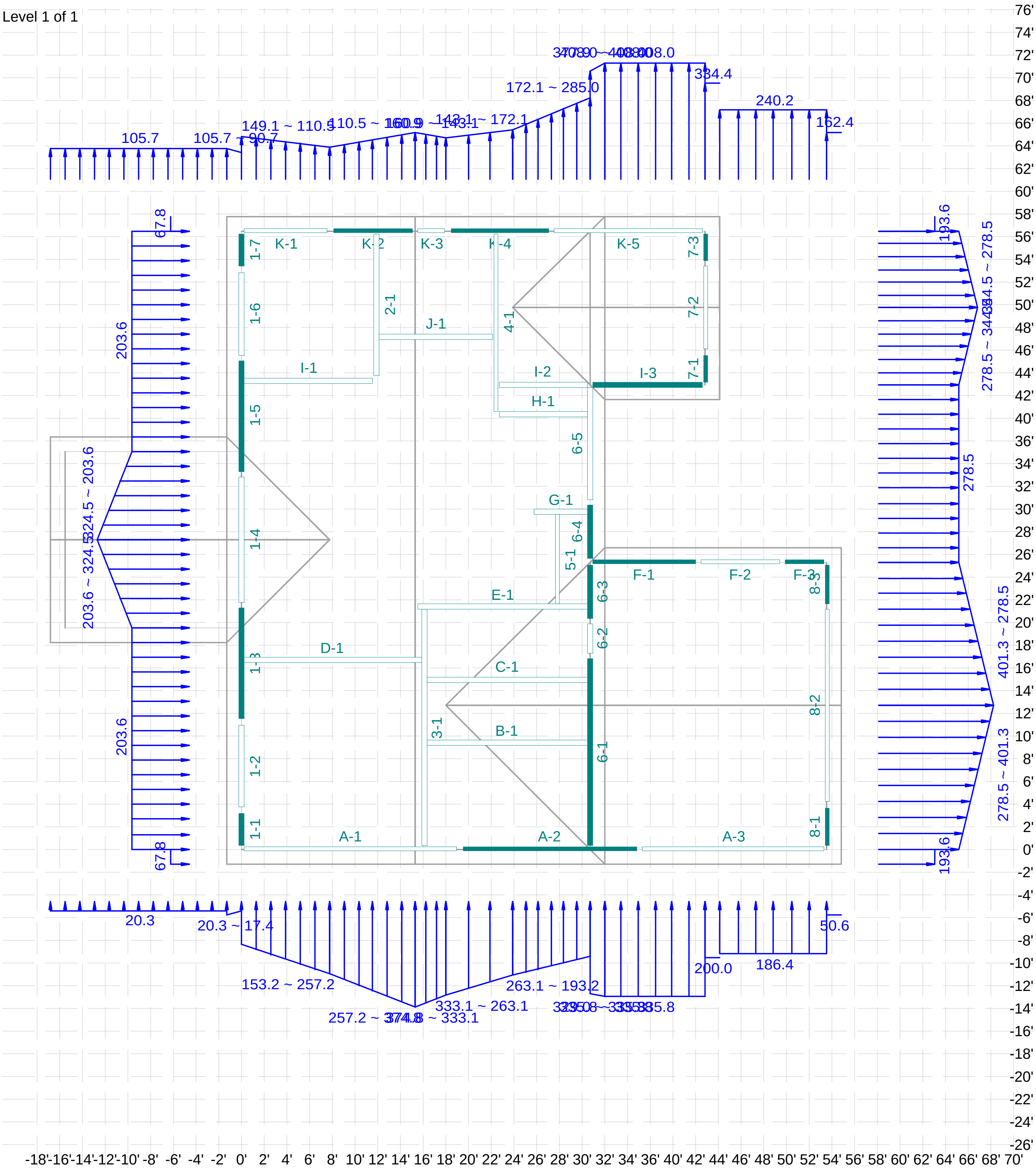
Location:	All 10' Garage Walls
Specification:	**Use 2X6 DF Stud Grade @ 16" o/c**

Rev. 1.3.1 - 06/23/2015

WoodWorks® Shearwalls

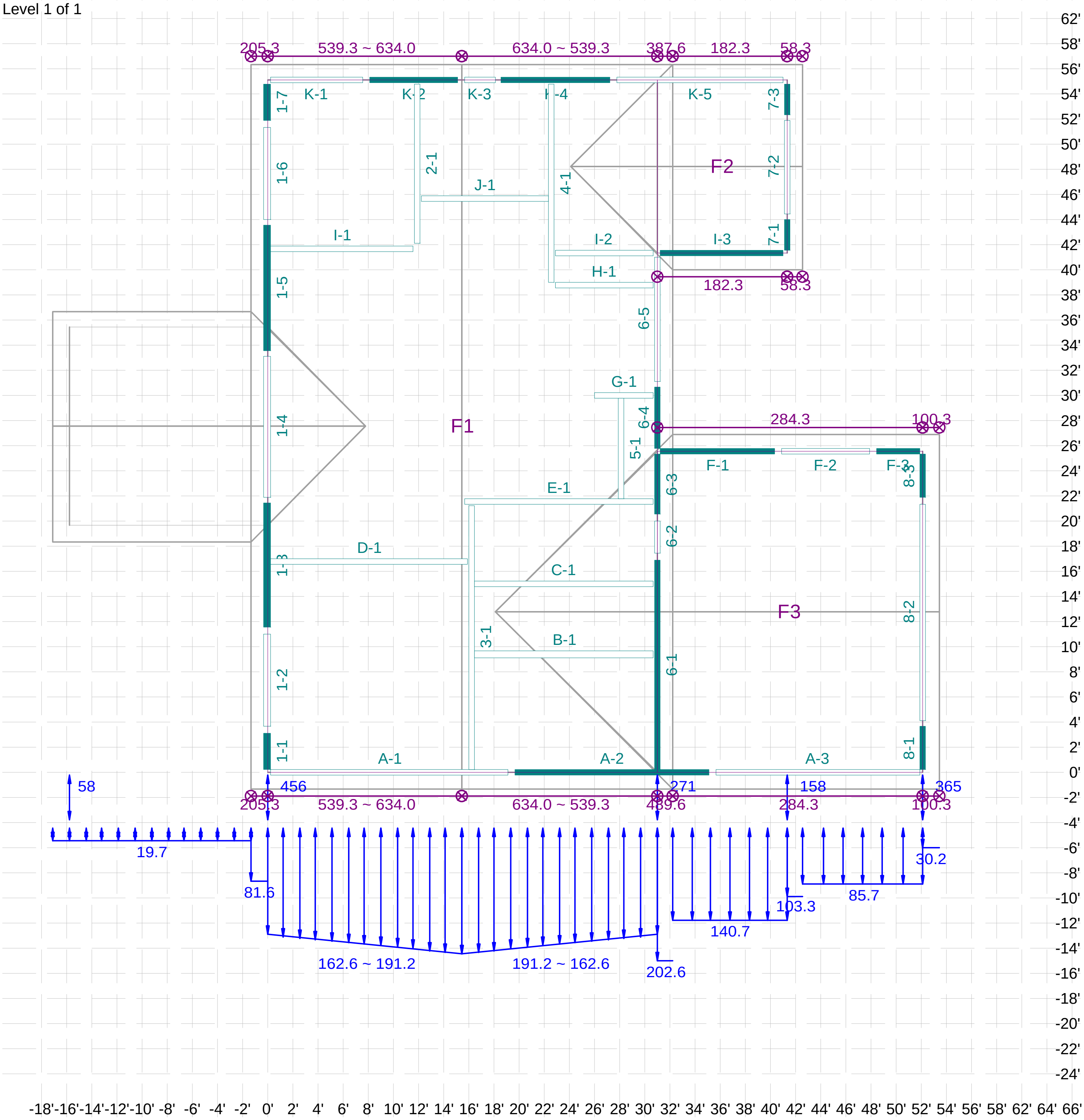
SOFTWARE FOR WOOD DESIGN

Level 1 of 1



Unfactored generated shear load (plf) Orange = Selected wall(s)

Level 1 of 1



Unfactored generated shear load (plf)

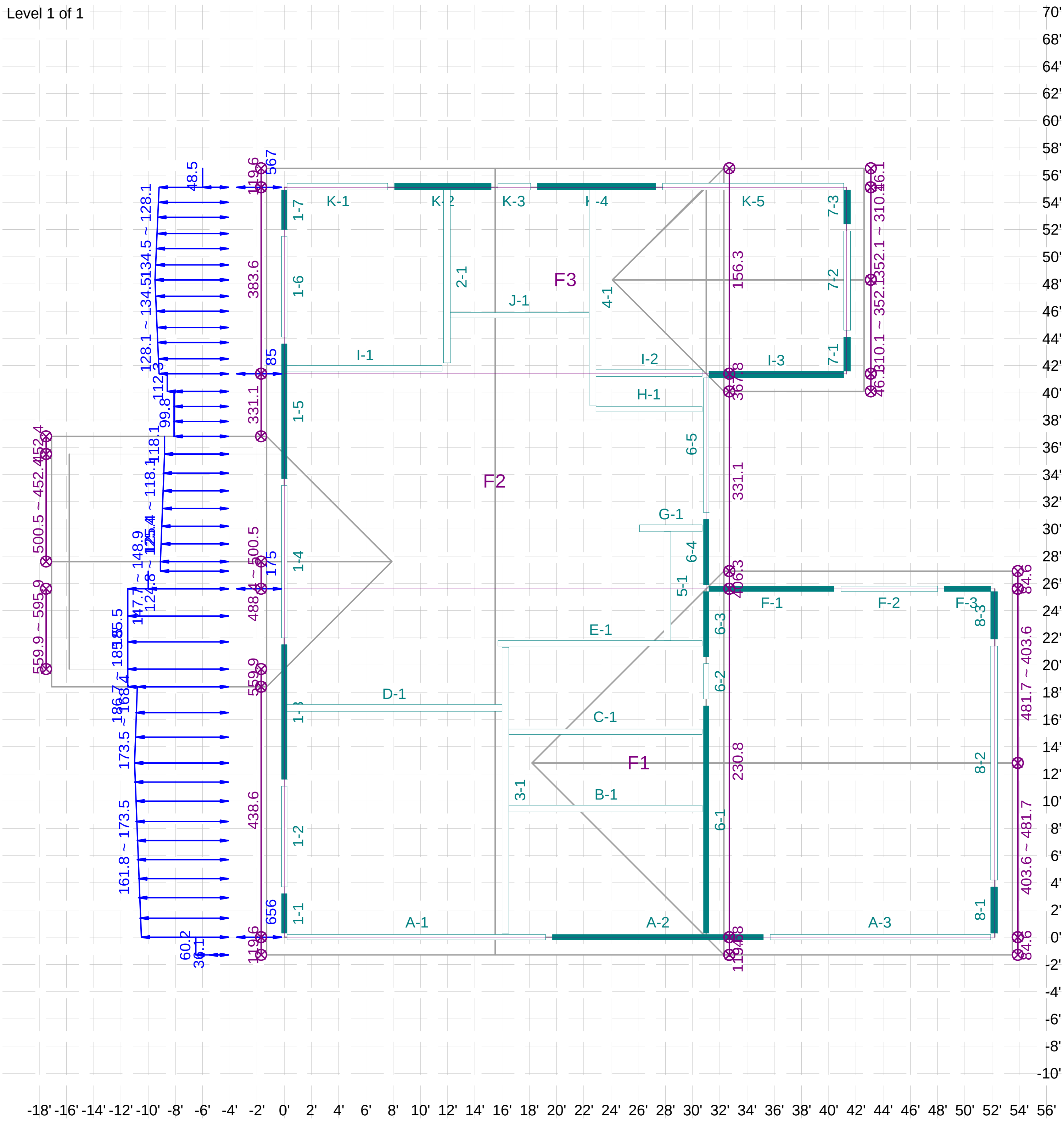
Generated building mass (plf,lbs)

F1 - Floor area 1 for mass generation

Generated point load from wall (lbs)

Orange = Selected wall(s)

Level 1 of 1


K-1
K-2
K-3
K-4
K-5
1-7
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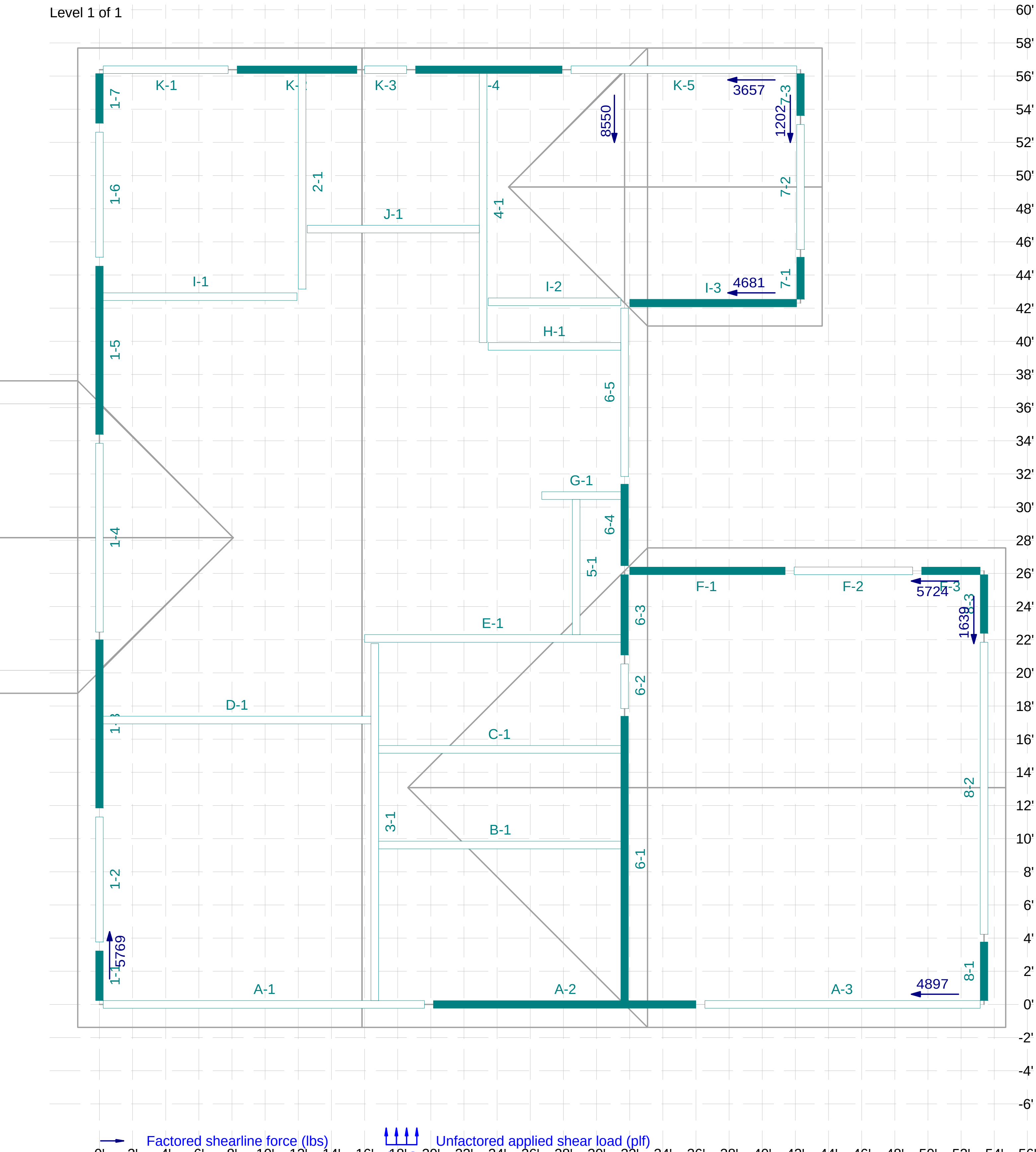
1-5
1-4
1-3
1-2
1-1
2-1
J-1
F3
4-1
7-3
7-2
7-1
I-1
I-2
I-3
H-1
6-5
F2
G-1
6-4
5-1
F-1
F-2
F-3
8-3
E-1
6-3
6-2
D-1
C-1
F1
8-2
3-1
B-1
6-1
8-1
A-1
A-2
A-3
Unfactored generated shear load (plf)
Generated building mass (plf,lbs)
F1 - Floor area 1 for mass generation
Generated point load from wall (lbs)
Orange = 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)

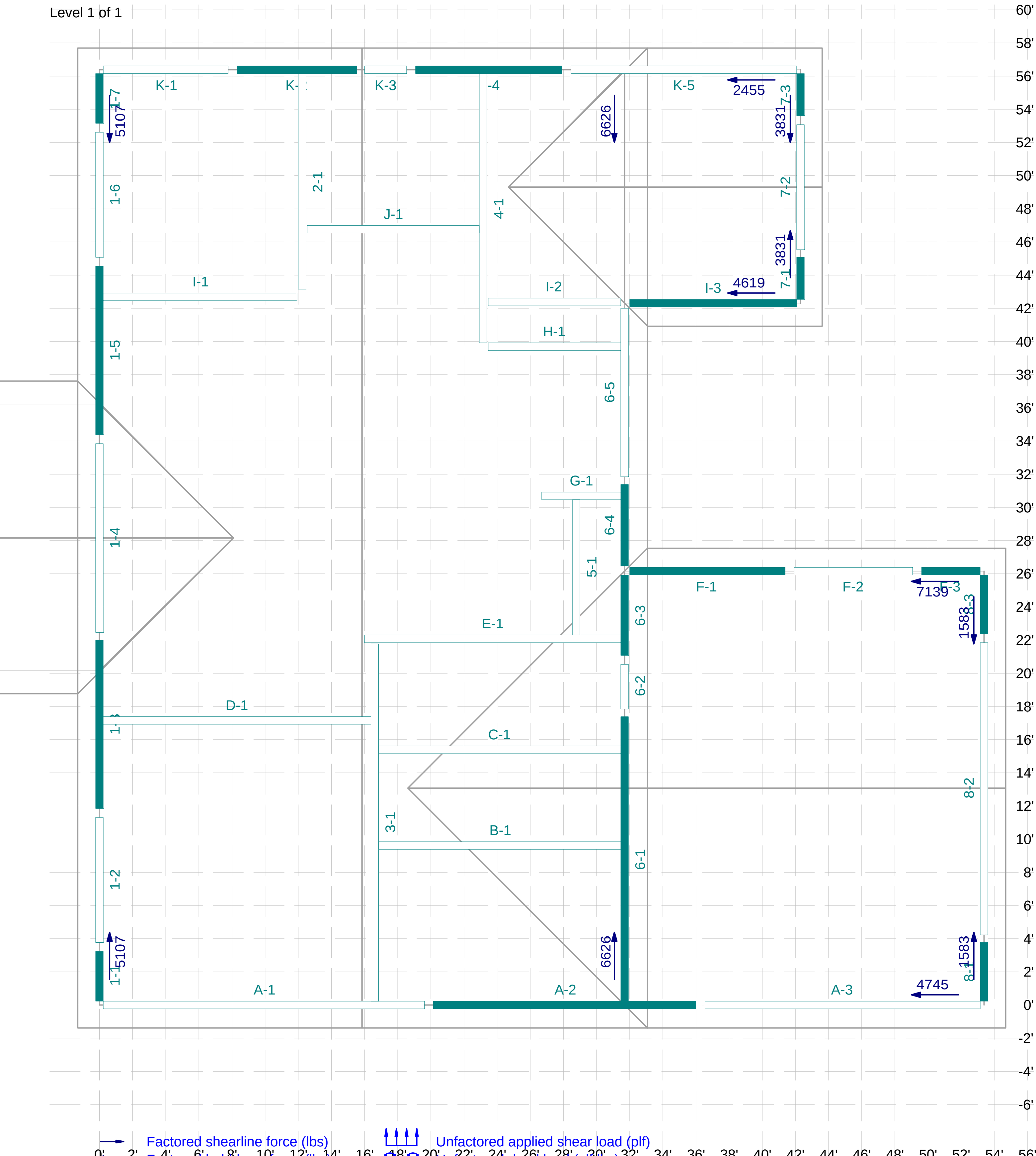
Unfactored dead load (plf,lbs)

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)

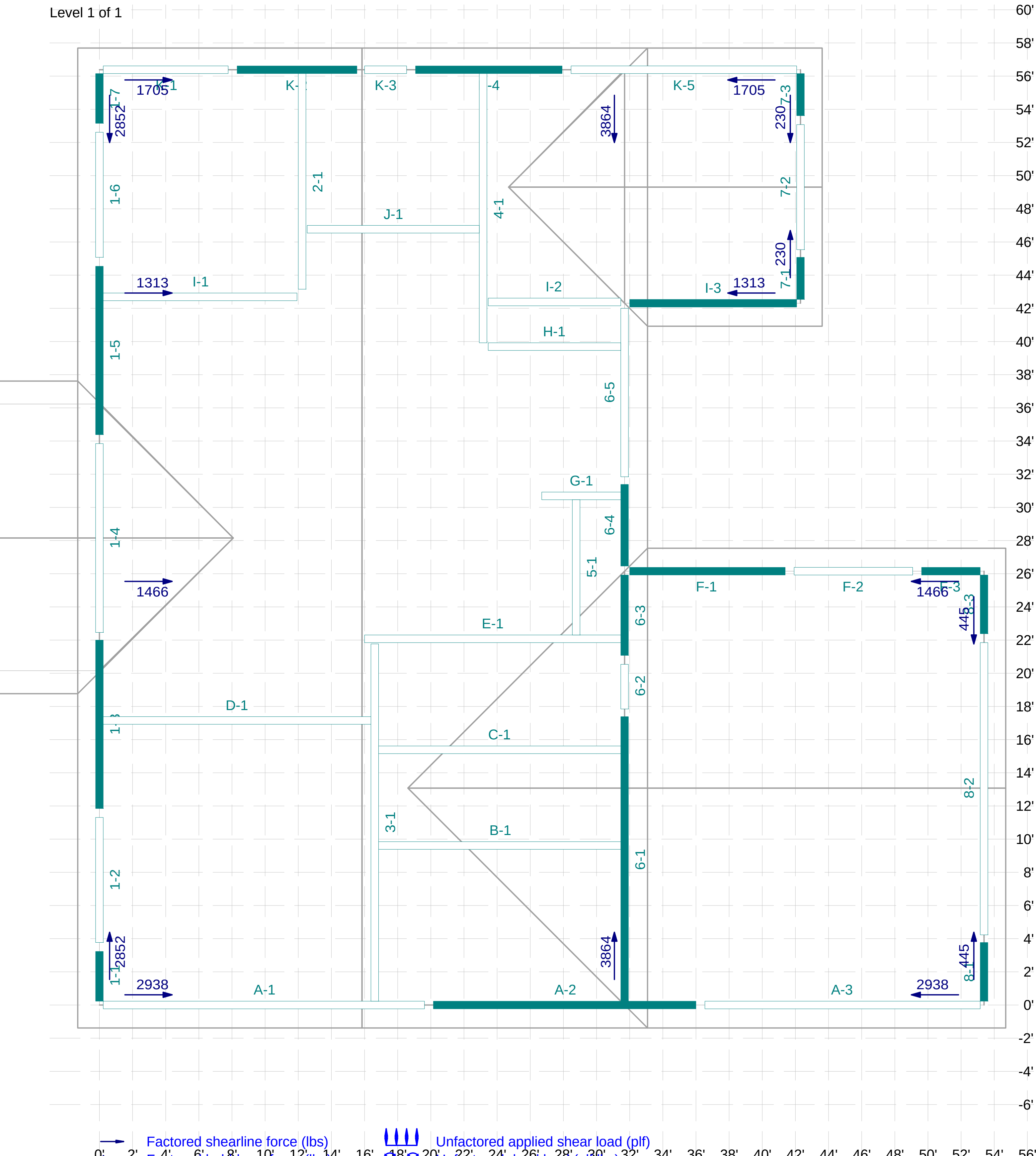
Level 1 of 1



Factored shearline force (lbs)
Factored holddown force (lbs)
C Compression force exists
Vertical element required
Loads Shown: W; Forces: 0.6W + 0.6D.

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

Orange = Selected wall(s)

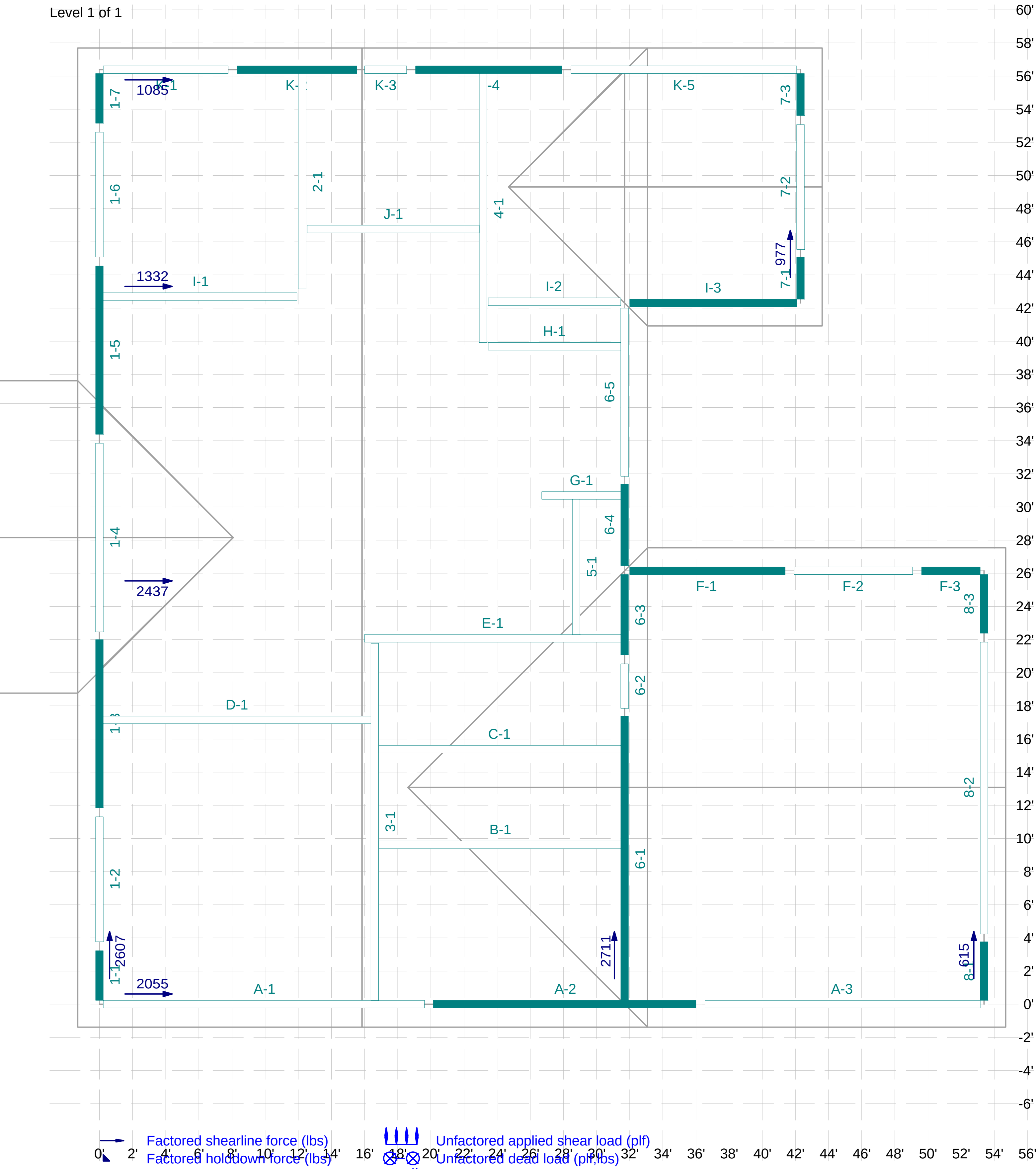


Factored shearline force (lbs)
Factored holddown force (lbs)
● C Compression force exists
■ Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: Qe; Forces: 0.7E + 0.6D; E = pQe + 0.2 Sds D; p(NS) = 1.0; p(EW) = 1.0; Sds = 0.98.

Orange = Selected wall(s)

Level 1 of 1

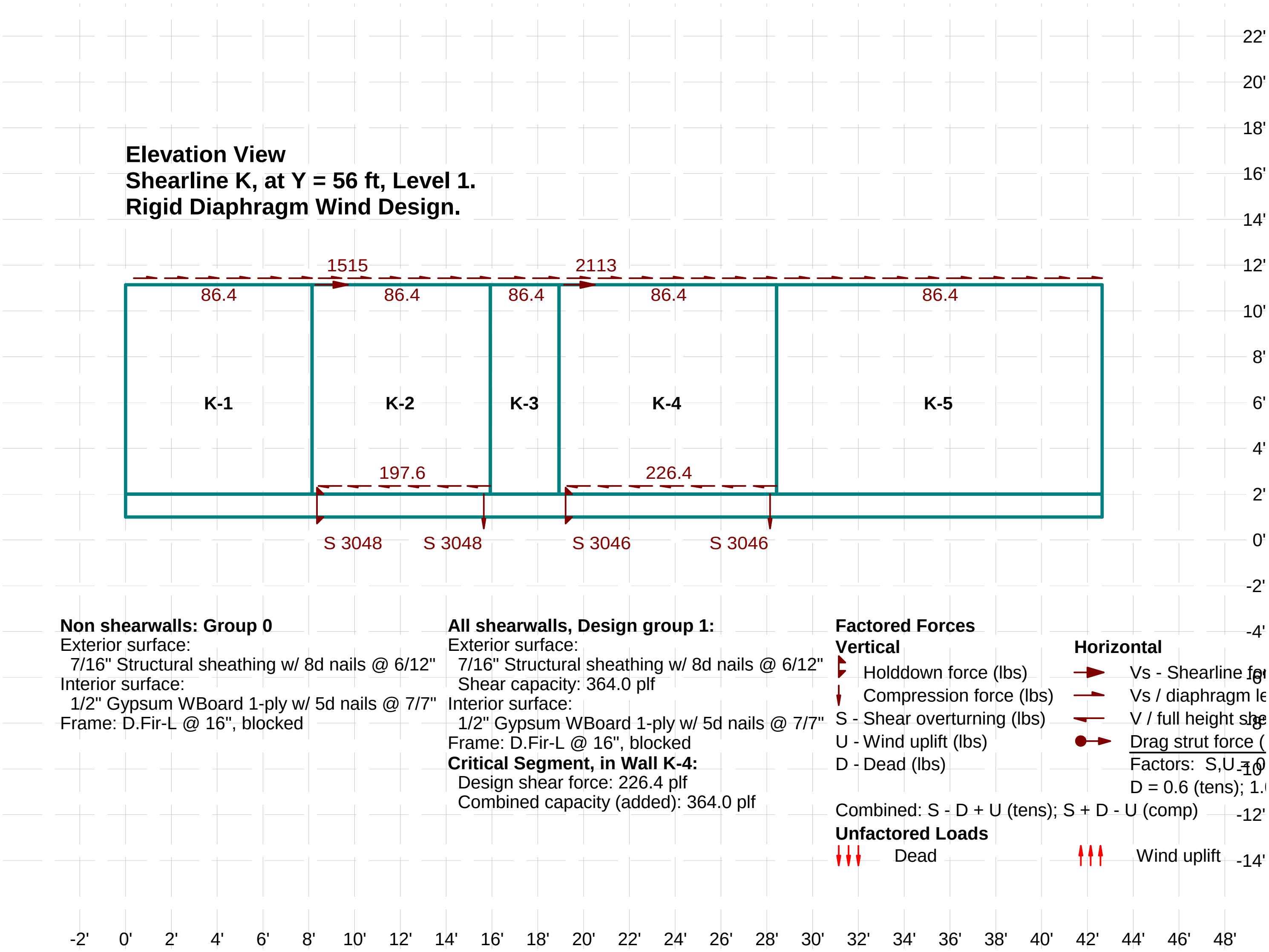


Factored shearline force (lbs)
Factored holddown force (lbs)
C Compression force exists
Vertical element required
Unfactored applied shear load (plf)
Unfactored dead load (plf,lbs)
Applied point load or discontinuous shearline force (lbs)

Loads Shown: Qe; Forces: 0.7E + 0.6D; E = pQe + 0.2 Sds D; p(NS) = 1.0; p(EW) = 1.0; Sds = 0.98.

Orange = Selected wall(s)

Elevation View
Shearline K, at Y = 56 ft, Level 1.
Rigid Diaphragm Wind Design.



Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 364.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall K-4:
Design shear force: 226.4 plf
Combined capacity (added): 364.0 plf

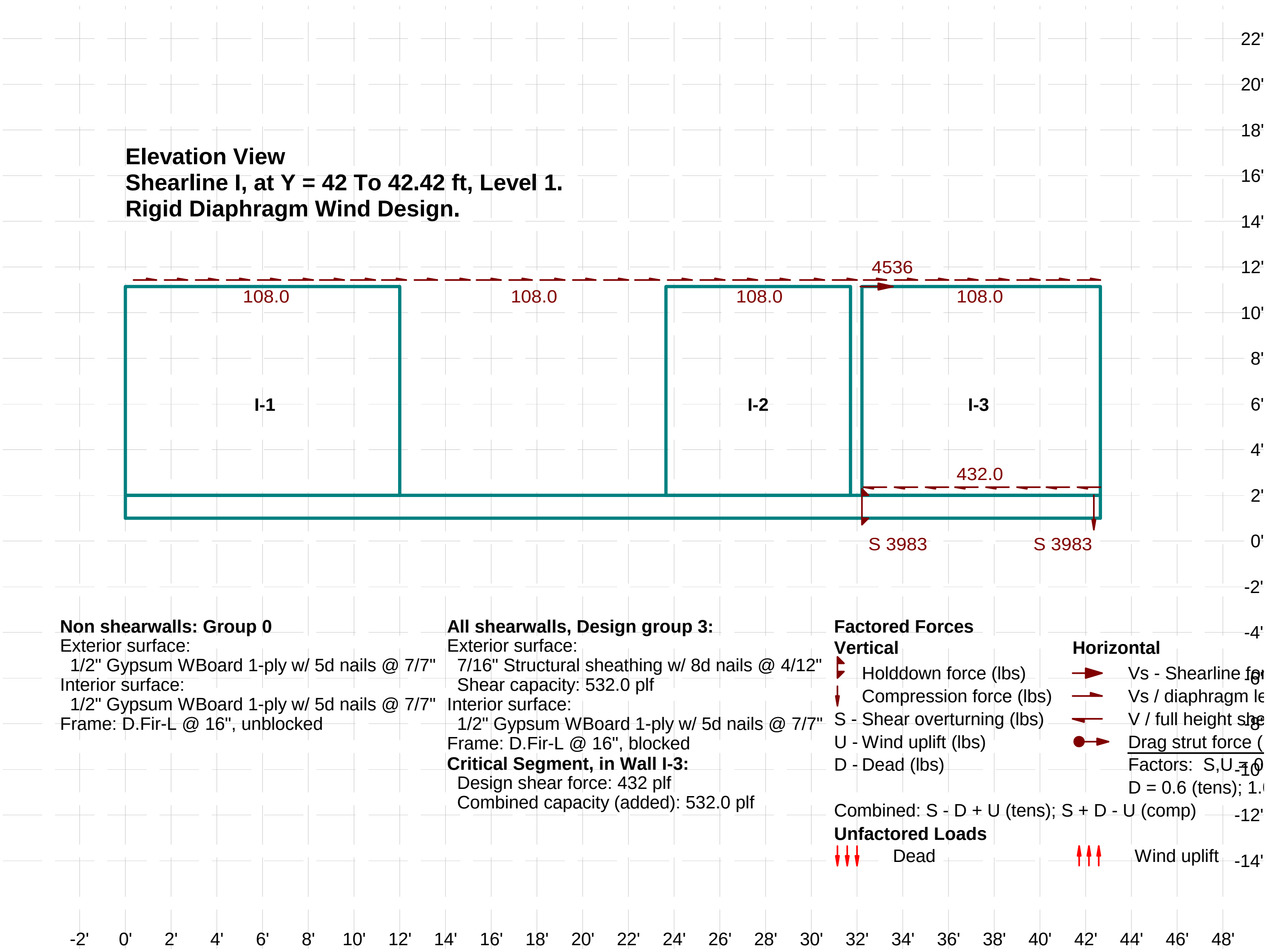
Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

West **East**

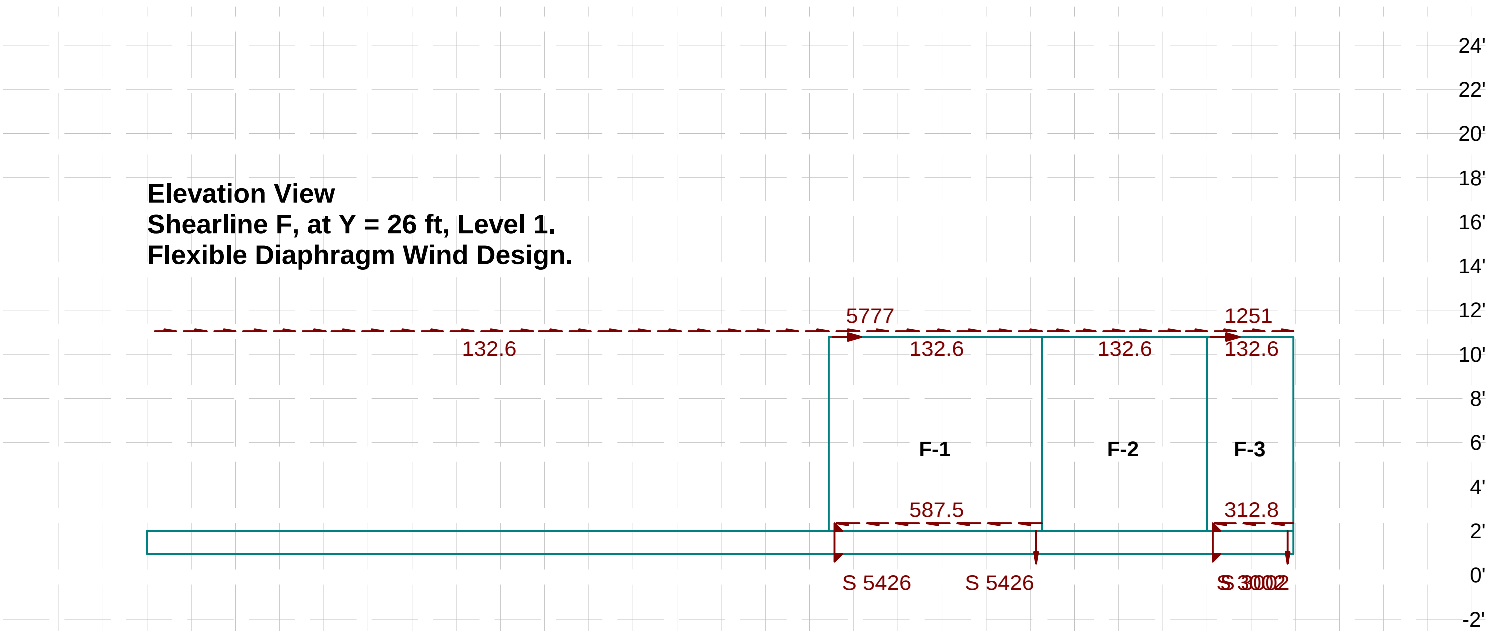
Elevation View
Shearline I, at Y = 42 To 42.42 ft, Level 1.
Rigid Diaphragm Wind Design.



West **East**

WoodWorks® Shearwalls — **SOFTWARE FOR WOOD DESIGN**

Elevation View
Shearline F, at Y = 26 ft, Level 1.
Flexible Diaphragm Wind Design.



All shearwalls, Design group 2:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 3/12"
Shear capacity: 686.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall F-1:
Design shear force: 587.5 plf
Combined capacity (added): 686.0 plf

Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

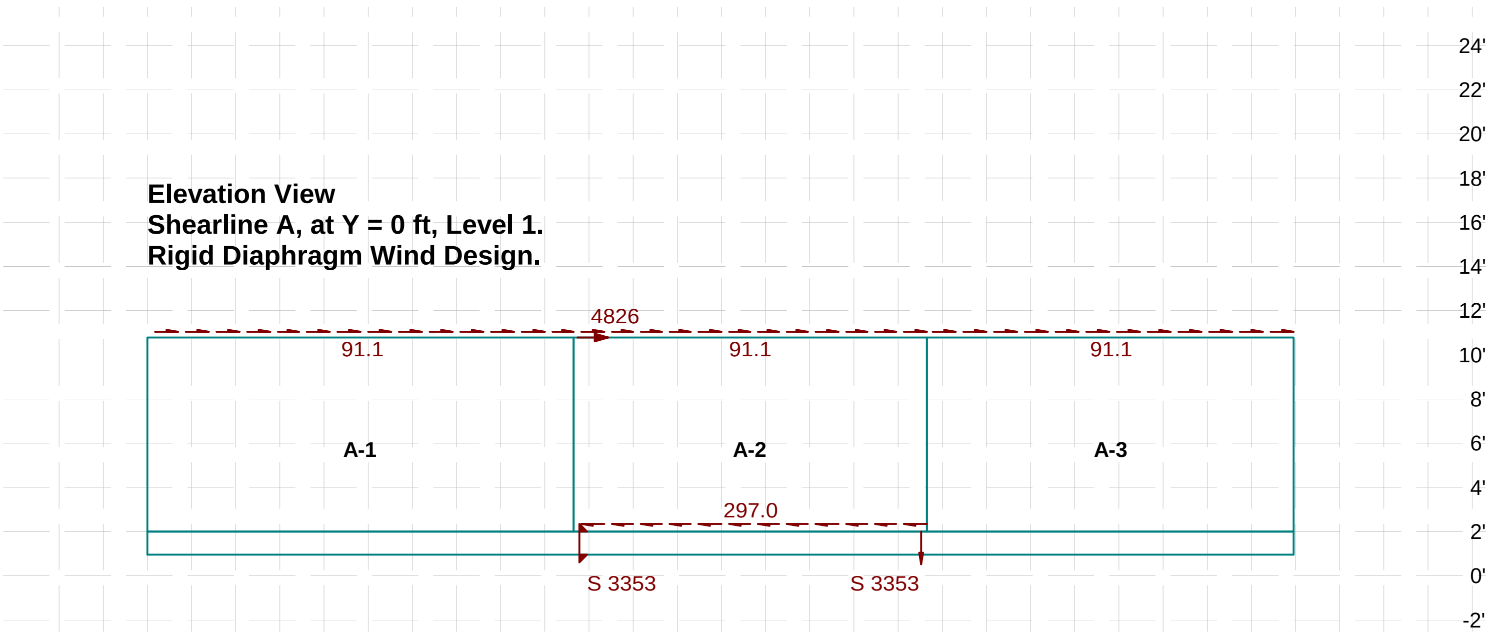
Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

-4' -2' 0' 2' 4' 6' 8' 10' 12' 14' 16' 18' 20' 22' 24' 26' 28' 30' 32' 34' 36' 38' 40' 42' 44' 46' 50' 52' 54' 56' 58' 60' 62'

-4' -6' -8' -10' -12' -14' -16' -18' -20'

West **East**

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



Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 364.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall A-2:
Design shear force: 297 plf
Combined capacity (added): 364.0 plf

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

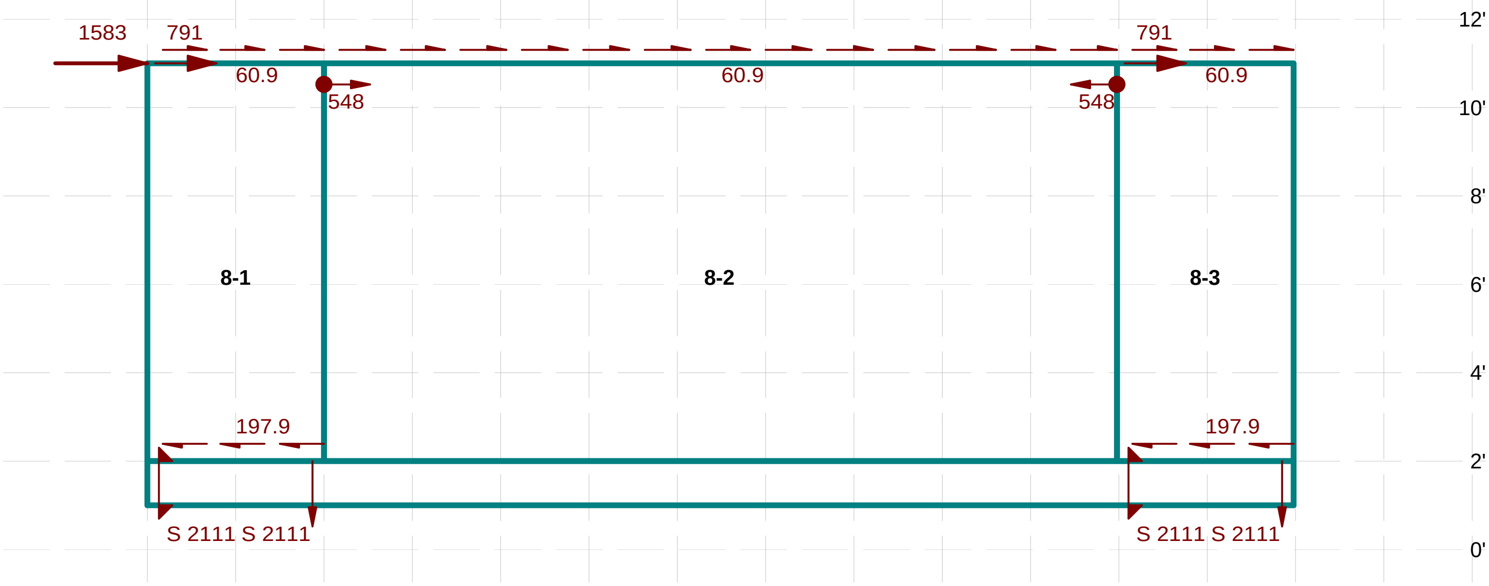
Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

-6' -8' -10' -12' -14' -16' -18' -20'

-4' -2' 0' 2' 4' 6' 8' 10' 12' 14' 16' 18' 20' 22' 24' 26' 28' 30' 32' 34' 36' 38' 40' 42' 44' 46' 50' 52' 54' 56' 58' 60' 62'

West **East**

Elevation View
Shearline 8, at X = 53 ft, Level 1.
Flexible Diaphragm Wind Design.



All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 364.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall 8-3:
Design shear force: 197.9 plf
Combined capacity (added): 364.0 plf

Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

-2' -4' -6' -8'

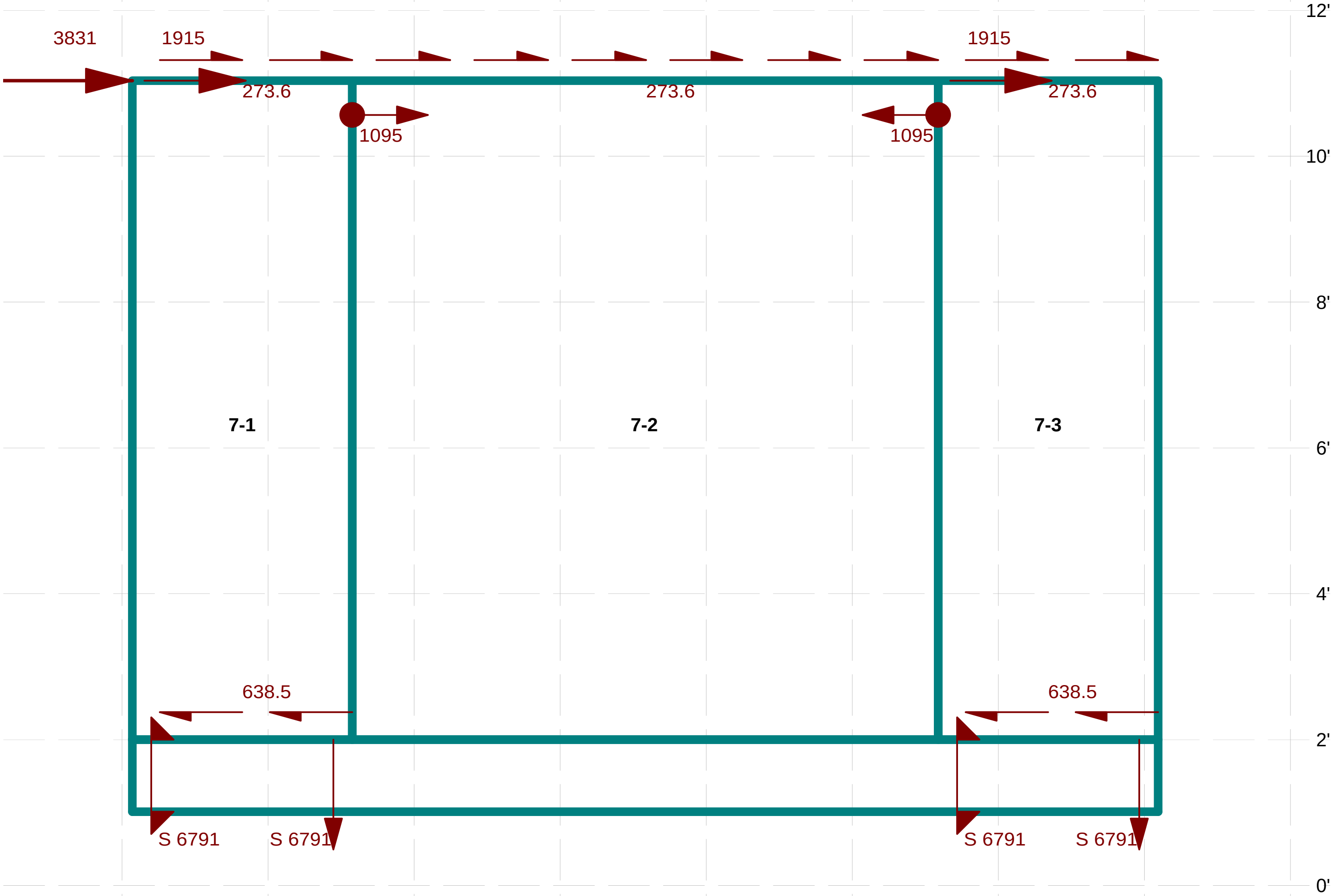
0' 2' 4' 6' 8' 10' 12' 14' 16' 18' 20' 22' 24' 26' 28' 30'

South **North**

WoodWorks® Shearwalls	**SOFTWARE FOR WOOD DESIGN**

2015-048_REV1.wsw **WoodWorks® Shearwalls 10.4** **Oct. 31, 2015 22:50:06**

Elevation View
Shearline 7, at X = 42 ft, Level 1.
Flexible Diaphragm Wind Design.



All shearwalls, Design group 2:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 3/12"
Shear capacity: 686.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall 7-1:
Design shear force: 638.5 plf
Combined capacity (added): 686.0 plf

Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

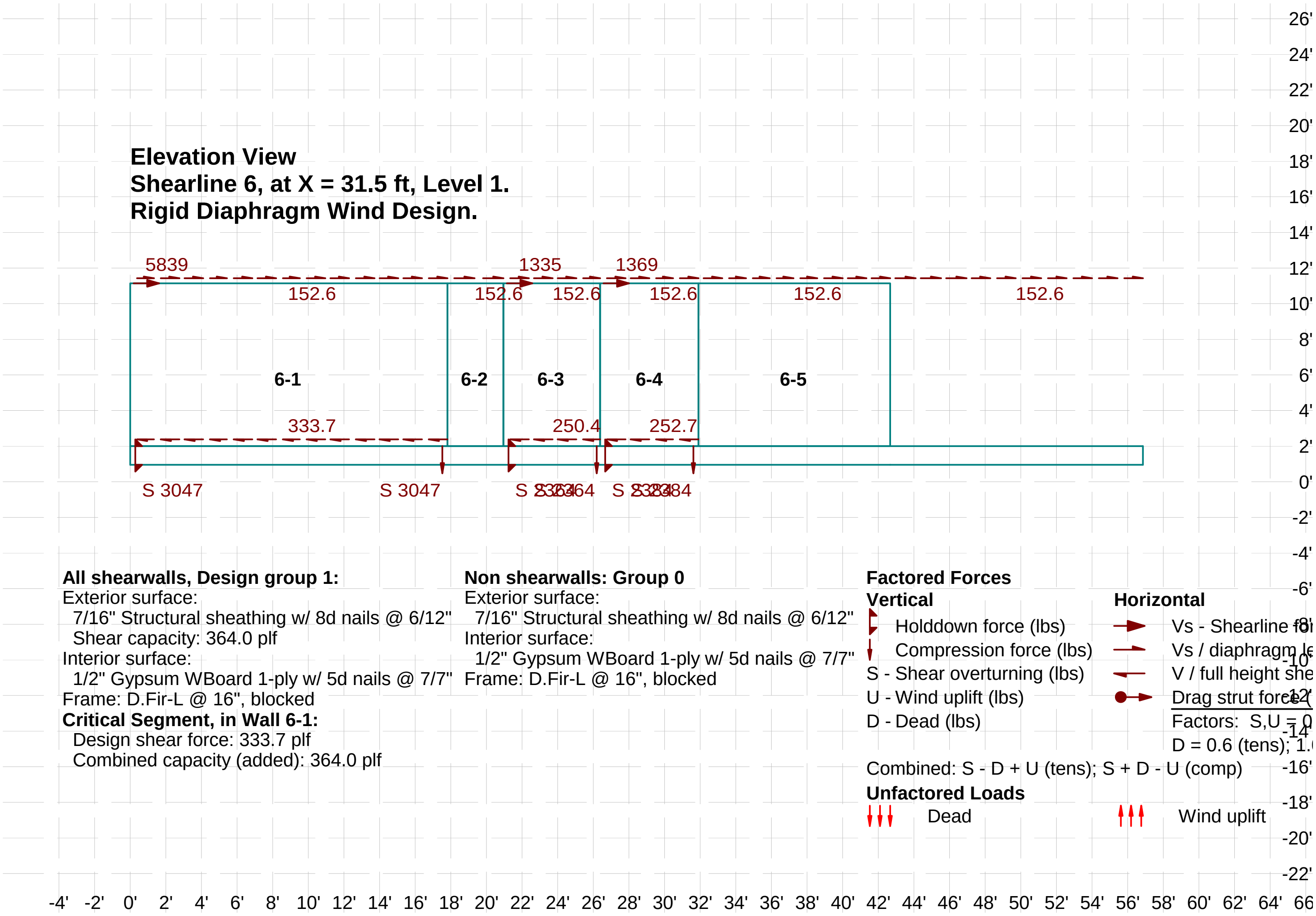
Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

-2'

South **North**

Elevation View
Shearline 6, at X = 31.5 ft, Level 1.
Rigid Diaphragm Wind Design.



All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 364.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall 6-1:
Design shear force: 333.7 plf
Combined capacity (added): 364.0 plf

Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

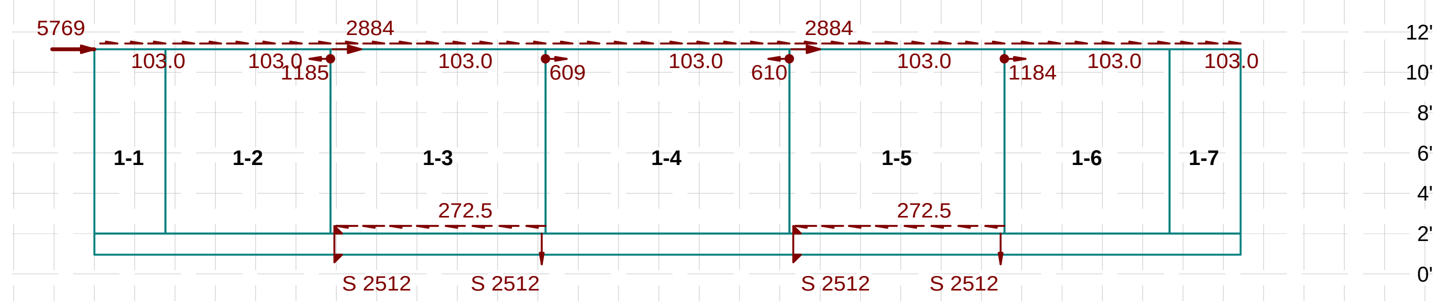
Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)

Unfactored Loads
Dead
Wind uplift

South **North**

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



All shearwalls, Design group 1:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Shear capacity: 364.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall 1-5:
Design shear force: 272.5 plf
Combined capacity (added): 364.0 plf

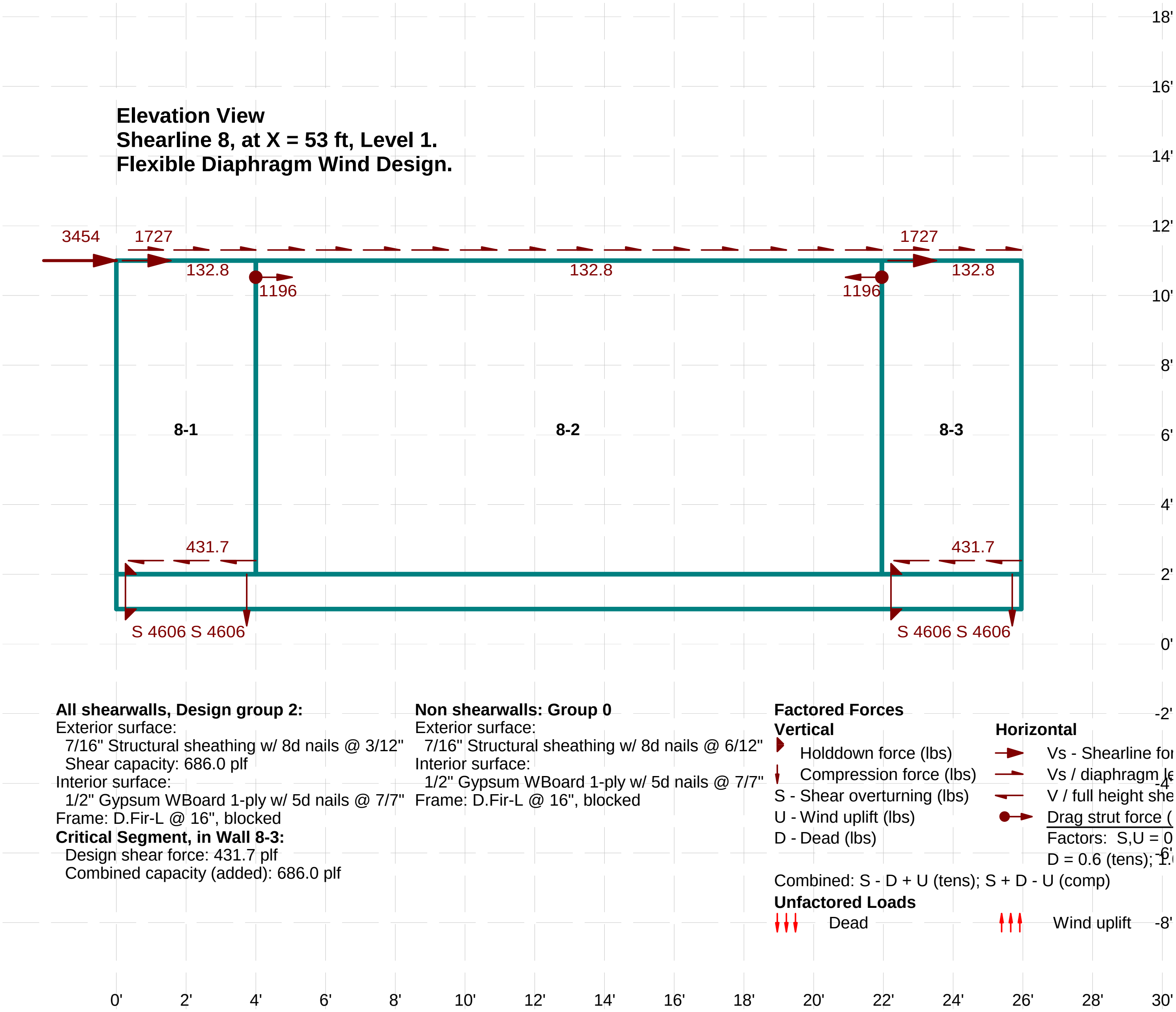
Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline fo
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

South **North**



All shearwalls, Design group 2:
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 3/12"
Shear capacity: 686.0 plf
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked
Critical Segment, in Wall 8-3:
Design shear force: 431.7 plf
Combined capacity (added): 686.0 plf

Non shearwalls: Group 0
Exterior surface:
7/16" Structural sheathing w/ 8d nails @ 6/12"
Interior surface:
1/2" Gypsum WBoard 1-ply w/ 5d nails @ 7/7"
Frame: D.Fir-L @ 16", blocked

Factored Forces
Vertical
Holddown force (lbs)
Compression force (lbs)
S - Shear overturning (lbs)
U - Wind uplift (lbs)
D - Dead (lbs)

Horizontal
Vs - Shearline for
Vs / diaphragm le
V / full height she
Drag strut force (
Factors: S,U = 0
D = 0.6 (tens); 1.

Combined: S - D + U (tens); S + D - U (comp)
Unfactored Loads
Dead
Wind uplift

South **North**

Job#: 2015-048

SHEARWALL SUMMARY

SWL	Wind Flex.	Wind Rigid	Wind Max.	Wind Avg.	Description
1	5,107	5,769	**5,769**	**5,438**	SEGMENTED
2	6,626	8,550	**8,550**	**7,588**	SEGMENTED
3	3,831	1,202	**3,831**	**2,517**	SEGMENTED
4	3,454	1,639	**3,454**	**2,547**	SEGMENTED
A	4,745	4,897	**4,897**	**4,821**	SEGMENTED
B	7,139	5,724	**7,139**	**6,432**	SEGMENTED
C	4,619	4,681	**4,681**	**4,650**	SEGMENTED
D	2,455	3,657	**3,657**	**3,056**	SEGMENTED

SWL	Seismic Flex.	Seismic Rigid	Seismic Max.	Seismic Avg.	Description
1	2,607	2,852	**2,852**	**2,730**	SEGMENTED
2	2,711	3,864	**3,864**	**3,288**	SEGMENTED
3	977	230	**977**	**604**	SEGMENTED
4	615	445	**615**	**530**	SEGMENTED
A	2,055	2,938	**2,938**	**2,497**	SEGMENTED
B	2,437	1,466	**2,437**	**1,952**	SEGMENTED
C	1,332	1,313	**1,332**	**1,323**	SEGMENTED
D	1,085	1,705	**1,705**	**1,395**	SEGMENTED

*Note: SWL4 Wind (Flexible) value based on secondary analysis with Shearwalls software.

Comments: Gable Trusses above SWLA, SWLD, SWL3 and SWL4 capable of lateral load from shearwall (sheathed).

Drag trusses inline with SWLB, SWLC capable of lateral load from shearwall (sheathed).

Drag truss above SWL2 capable of lateral load from shearwall (sheathed).

SHEAR WALL CALCULATOR **SWLD** Vs = 1705 lbs (seismic) Vw = 3657 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWLD	3,657	42.0	17.0	215.1	9.0	1,936	HDU2	SSTB16	12	(2) 2x6	2
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 = **100** plf < v_{allow} = 240 plf ⟶ OK (seismic)

v_w = **215** 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 = 7.625 ft
Max. AR: h/bs = **1.18** ⟶ 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 9.0 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 622 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 4.5 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 4,331 lbs

Wind Load Governs:
V_{perp} = 4,331 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	4,331	9.3	**4.5**
Para. Load	3,657	4.3	**9.7**

La = 42.0 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 D.

A35 Framing Angle Spacing

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

Lac = 17.0 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 215.1 plf
Spacing = **2.8** ft

Use A35 clips for top plt./blocking connection @ 32" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 140.4 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.088 in (Simpson Holdown)
en = 0.0017 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	7.625	0.13	in
2	9.375	0.11	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.13**	in

ASCE 7-10 (Table 12.12-1)

Cd = 4
Δ = 0.54 in
Δlimit = 2.16 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#: **2015-048**

<u>Bearing on Wall Plates</u>

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	2055	lbs
$f_{c\perp}$	125	psi
CSI (bearing)	**0.31**	→ **OK**

<u>Chord in Tension</u> (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	(wind)
C_{F_t}	1.30	
F_t'	1196	psi
A_n	16.50	in^2
f_t	117	psi
CSI (tension)	**0.10**	→ **OK**

<u>Chord in Compression</u> (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	(wind)
C_{F_c}	1.10	
$(l_e/d)_x$	18.82	
E'_{min}	580,000	psi
F_{cE}	1346	psi
F_c^*	2376	psi
c	0.80	sawn lumber
F_{cE}/F_c^*	0.567	
$1 + F_{cE}/F_c^*/2c$	0.979	
C_p	0.479	
F_c'	1137	psi
f_c	125	psi
CSI (compression)	**0.11**	→ **OK**

<u>Shearwall Gravity Loads</u> (Point loads are assumed to bear directly above SWL chord)

(plf)	W_{DL}	W_{LL}	W_{SL}/W_{LrL}
Wall Loads	89	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 =	2,055	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	1,616	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,021	plf
6b.) D + .75L + .75E + 75S =	841	plf

P_W =	1,936	lbs
P_S =	903	lbs
* SWL Chord Tension =	**1,936**	lbs
SWL Chord Comp. =	**2,055**	lbs
Stud Spacing =	16	in
Chord Studs =	(2) 2x6	
Chord Depth (dx) =	5.5	in
l_b =	3.00	in

<u>Bottom Plate (Sole Plt.) Attachment to Floor</u>

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 =	215.1	plf
Spacing =	**12.6**	in

Emin =	580,000	psi
C_{M_e} =	1.00	
C_{t_e} =	1.00	
C_{t_e} =	1.00	

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

<u>Sill Plate at Foundation</u>

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

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWLC** Vs = 1332 lbs (seismic) Vw = 4681 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWLC	4,681	10.5	10.5	445.8	9.0	4,012	HDU4	SB5/8X24	18	(3) 2x6	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 = **127** plf < v_{allow} = 350 plf → OK (seismic)

v_w = **446** plf < v_{allow} = 490 plf → OK (wind)

Edge Nail Spacing = 4 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 10.5 ft
Max. AR: h/bs = **0.86** → OK
Max. AR Seismic Reduction: 2bs/h = N/A

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 4" o/c edges, 12" o/c field, blocking required. Members and blocking at adjoining panel edges shall be min. 3" nominal or double 2" nominal with staggered nailing at all panel edges.

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 = 0.625 in
Zpara = 780 lbs
Zperp = 320 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 1248 lbs
Zperp = 512 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 9.0 ft
ρ = 1.0 (out-of-plane)

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

V_{sperp} = 155 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 4.5 ft

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

V_{wperp} = 1,083 lbs

Wind Load Governs:

V_{perp} = 1,083 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	1,083	2.1	**5.0**
Para. Load	4,681	3.8	**2.8**

La = 10.5 ft La = available wall length for anchor bolts

Use 5/8" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 32" o/c spacing all of Wall C.

A35 Framing Angle Spacing

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

Lac = 10.5 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 445.8 plf
Spacing = **1.3** ft

Use A35 clips for top plt./blocking connection @ 16" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 177.6 plf
E = 1,600,000 psi
A = 24.75 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.114 in (Simpson Holdown)
en = 0.0010 in (Table C4.2.2D)
nail spacing = 4 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	10.5	0.13	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.13**	in

Cd = 4
ASCE 7-10 Δ = 0.50 in
(Table 12.12-1) Δlimit = 2.16 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.

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
$F_{c\perp}$	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.083)
$F_{c\perp}'$	405.00	psi
Ab	24.75	in^2
Pc	7602	lbs
$f_{c\perp}$	307	psi
CSI (bearing)	**0.76**	→ **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	1.30	
Ft'	1196	psi
An	24.75	in^2
ft	162	psi
CSI (tension)	**0.14**	→ **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	1.10	
(le/d)x	18.82	
E'min	580,000	psi
FcE	1346	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.567	
1 + FcE/Fc*/2c	0.979	
Cp	0.479	
Fc'	1137	psi
fc	307	psi
CSI (compression)	**0.27**	→ **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	135	0	194

(lbs)	P_{DL}	P_{LL}	P_{SL}/P_{LrL}	P_W (+/-)	P_S (+/-)
Point loads	2,492	0	2,303	0	0

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	6,684	plf
6a.) D + .75L + .75W + 75(Lr or S) =	7,602	plf (governs)

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	3,814	plf
6b.) D + .75L + .75E + 75S =	5,450	plf

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Pw =	4,012	lbs
Ps =	1,142	lbs
* SWL Chord Tension =	**4,012**	lbs
SWL Chord Comp. =	**7,602**	lbs
Stud Spacing =	16	in
Chord Studs =	(3) 2x6	
Chord Depth (dx) =	5.5	in
lb =	4.50	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 =	445.8	plf
Spacing =	**6.1**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

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 = 2437 lbs (seismic) Vw = 7139 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWLB	7,139	21.5	13.8	516.1	10.0	5,161	HDU5	SB5/8X24	18	(2) 2x6	2
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 = **176** plf < v_{allow} = 424 plf ⟶ OK (seismic)

v_w = **516** plf < v_{allow} = 630 plf ⟶ OK (wind)

Edge Nail Spacing = 3 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 4 ft
Max. AR: h/bs = **2.50** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = 0.80

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 3" o/c edges, 12" o/c field, blocking required. Members and blocking at adjoining panel edges shall be min. 3" nominal or double 2" nominal with staggered nailing at all panel edges.

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 = 0.625 in
Zpara = 780 lbs
Zperp = 320 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 1248 lbs
Zperp = 512 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 10.0 ft
ρ = 1.0 (out-of-plane)

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

V_{sperp} = 354 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 5 ft

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

V_{wperp} = 2,463 lbs

Wind Load Governs:

V_{perp} = 2,463 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	2,463	4.8	**3.8**
Para. Load	7,139	5.7	**3.2**

La = 18.5 ft La = available wall length for anchor bolts

Use 5/8" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 36" o/c spacing all of Wall B.

A35 Framing Angle Spacing

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

Lac = 21.5 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 332.0 plf
Spacing = **1.8** ft

Use A35 clips for top plt./blocking connection @ 18" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 246.6 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.115 in (Simpson Holdown)
en = 0.0012 in (Table C4.2.2D)
nail spacing = 3 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	9.833	0.16	in
2	4	0.34	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.34**	in

Cd = 4
ASCE 7-10 Δ = 1.38 in
(Table 12.12-1) Δlimit = 2.4 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#: 2015-048

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	16.50	in^2
Pc	5484	lbs
fc⊥	332	psi
CSI (bearing)	**0.82**	→ **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	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	313	psi
CSI (tension)	**0.26**	→ **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	1.10	
(le/d)x	21.00	
E'min	580,000	psi
FcE	1081	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.455	
1 + FcE/Fc*/2c	0.909	
Cp	0.401	
Fc'	953	psi
fc	332	psi
CSI (compression)	**0.35**	→ **OK**

Shearwall Gravity Loads

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

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	242	0	282

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

Pw = 5,161 lbs

Ps = 1,762 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	5,484	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	4,475	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	2,084	plf
6b.) D + .75L + .75E + 75S =	1,926	plf

* SWL Chord Tension =	**5,161**	lbs
SWL Chord Comp. =	**5,484**	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 =	516.1	plf
Spacing =	**5.2**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

Sill Plate at Foundation

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

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR | **SWLA** | Vs = 2938 lbs (seismic) | Vw = 4897 lbs (wind) | **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWLA	4,897	53.0	16.3	301.4	10.0	3,014	HDU4	SB5/8X24	18	(2) 2x6	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 = **181** plf < v_{allow} = 240 plf → OK (seismic)

v_w = **301** 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 = 16.25 ft
Max. AR: h/bs = **0.62** → 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 10.0 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 872 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 5 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 6,072 lbs

Wind Load Governs:
V_{perp} = 6,072 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	6,072	13.1	**4.0**
Para. Load	4,897	5.8	**9.2**

La = 53.0 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 A.

A35 Framing Angle Spacing

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

Lac = 16.3 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 301.4 plf
Spacing = **2.0** ft

Use A35 clips for top plt./blocking connection @ 24" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 253.1 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.114 in (Simpson Holdown)
en = 0.0101 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	16.25	0.18	in
2	0	--	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.18**	in

ASCE 7-10 (Table 12.12-1)

Cd = 4
Δ = 0.72 in
Δlimit = 2.4 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#: **2015-048**

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
$F_{c\perp}$	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.125)
$F_{c\perp}'$	405.00	psi
Ab	16.50	in^2
Pc	3336	lbs
$f_{c\perp}$	202	psi
CSI (bearing)	**0.50**	→ **OK**

Chord in Tension

(DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
C_D	1.60	(wind)
CF_t	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	183	psi
CSI (tension)	**0.15**	→ **OK**

Chord in Compression

(DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
C_D	1.60	(wind)
CF_c	1.10	
(le/d)x	21.00	
E'min	580,000	psi
F_{cE}	1081	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.455	
1 + FcE/Fc*/2c	0.909	
Cp	0.401	
Fc'	953	psi
fc	202	psi
CSI (compression)	**0.21**	→ **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	242	0	282

(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 =	3,336	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	2,865	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	2,131	plf
6b.) D + .75L + .75E + 75S =	1,961	plf

P_W =	3,014	lbs
P_S =	1,808	lbs
* SWL Chord Tension =	**3,014**	lbs
SWL Chord Comp. =	**3,336**	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 =	301.4	plf
Spacing =	**9.0**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

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 = 615 lbs (seismic) Vw = 3454 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL4	3,454	26.0	8.0	431.8	10.0	4,318	HDU5	SB5/8X24	18	(2) 2x6	2
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 = **77** plf < v_{allow} = 360 plf ⟶ OK (seismic)

v_w = **432** plf < v_{allow} = 630 plf ⟶ OK (wind)

Edge Nail Spacing = 3 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 4 ft
Max. AR: h/bs = **2.50** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = 0.80

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 3" o/c edges, 12" o/c field, blocking required. Members and blocking at adjoining panel edges shall be min. 3" nominal or double 2" nominal with staggered nailing at all panel edges.

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: (2)-2x
AB DIA = 0.625 in
Zpara = 940 lbs
Zperp = 460 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 1504 lbs
Zperp = 736 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 10.0 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 428 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 5 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 2,979 lbs

Wind Load Governs:
V_{perp} = 2,979 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	2,979	4.0	**2.0**
Para. Load	3,454	2.3	**3.5**

La = 8.0 ft La = available wall length for anchor bolts

Use 5/8" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 24" o/c spacing all of Wall 4.

A35 Framing Angle Spacing

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

Lac = 8.0 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 431.8 plf
Spacing = **1.4** ft

Use A35 clips for top plt./blocking connection @ 16" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 107.6 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.115 in (Simpson Holdown)
en = 0.0001 in (Table C4.2.2D)
nail spacing = 3 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	4	0.31	in
2	4	0.31	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.31**	in

Cd = 4
ASCE 7-10 (Table 12.12-1)
Δ = 1.24 in
Δlimit = 2.4 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.

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	16.50	in^2	
Pc	4426	lbs	
fc⊥	268	psi	
CSI (bearing)	**0.66**	→	**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	1.30		
Ft'	1196	psi	
An	16.50	in^2	
ft	262	psi	
CSI (tension)	**0.22**	→	**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	1.10		
(le/d)x	21.00		
E'min	580,000	psi	
FcE	1081	psi	
Fc*	2376	psi	
c	0.80	sawn lumber	
FcE/Fc*	0.455		
1 + FcE/Fc*/2c	0.909		
Cp	0.401		
Fc'	953	psi	
fc	268	psi	
CSI (compression)	**0.28**	→	**OK**

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

Job#: **2015-048**

(plf)	W_{DL}	W_{LL}	W_{SL}/W_{LrL}
Wall Loads	81	0	45

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

Pw =	4,318	lbs
Ps =	769	lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	4,426	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	3,391	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	877	plf
6b.) D + .75L + .75E + 75S =	730	plf

* SWL Chord Tension =	**4,318**	lbs
SWL Chord Comp. =	**4,426**	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 =	431.8	plf
Spacing =	**6.3**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

Sill Plate at Foundation

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

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL3** Vs = 977 lbs (seismic) Vw = 2517 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL3	2,517	14.0	6.0	419.5	9.0	3,776	HDU4	SB5/8X24	18	(2) 2x6	2
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 = **163** plf < v_{allow} = 233 plf ⟶ OK (seismic)

v_w = **420** plf < v_{allow} = 490 plf ⟶ OK (wind)

Edge Nail Spacing = 4 in
Sheathing both sides = NO
Sht. Panel Thickness = 7/16 in
Fastener Type = 8d
Min. Panel Length: bs = 3 ft
Max. AR: h/bs = **3.00** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = 0.67

Use 7/16 OSB/PLY (APA Grade 24/16) w/ 8d nails @ 4" o/c edges, 12" o/c field, blocking required. Members and blocking at adjoining panel edges shall be min. 3" nominal or double 2" nominal with staggered nailing at all panel edges.

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 = 0.625 in
Zpara = 780 lbs
Zperp = 320 lbs
Applying adjustment factors:
CD = 1.6 (wind or seismic)
Zpara = 1248 lbs
Zperp = 512 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 9.0 ft
ρ = 1.0 (out-of-plane)
Vsperp is given as the seismic force of half the dead weight of the wall.
V_{sperp} = 207 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 4.5 ft
Vwperp is given as the max. MWFRS wind force on the bottom half of an exterior wall.
V_{wperp} = 1,444 lbs

Wind Load Governs:
V_{perp} = 1,444 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	1,444	2.8	**2.1**
Para. Load	2,517	2.0	**3.0**

La = 6.0 ft La = available wall length for anchor bolts

Use 5/8" DIA anchor bolts, 7" min. embedment /w 3"x3"x1/4" washers @ 24" o/c spacing all of Wall 3.

A35 Framing Angle Spacing

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

Lac = 6.0 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 419.5 plf
Spacing = **1.4** ft

Use A35 clips for top plt./blocking connection @ 16" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 228.0 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.114 in (Simpson Holdown)
en = 0.0022 in (Table C4.2.2D)
nail spacing = 4 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	3	0.40	in
2	3	0.40	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.40**	in

Cd = 4
ASCE 7-10 (Table 12.12-1)
Δ = 1.59 in
Δlimit = 2.16 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.

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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	
Cb	1.00	(1.125)
$F_{c\perp}'$	405.00	psi
Ab	16.50	in^2
Pc	4175	lbs
$f_{c\perp}$	253	psi
CSI (bearing)	**0.62**	→ **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	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	229	psi
CSI (tension)	**0.19**	→ **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	1.10	
(le/d)x	18.82	
E'min	580,000	psi
FcE	1346	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.567	
1 + FcE/Fc*/2c	0.979	
Cp	0.479	
Fc'	1137	psi
fc	253	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	63	0	45

(lbs)	PDL	PLL	PSL/PLrL	PW (+/-)	PS (+/-)
Point loads	315	0	193	0	0

Pw = 3,776 lbs
Ps = 1,466 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	4,175	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	3,420	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,865	plf
6b.) D + .75L + .75E + 75S =	1,688	plf

* SWL Chord Tension =	**3,776**	lbs
SWL Chord Comp. =	**4,175**	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 =	419.5	plf
Spacing =	**6.5**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

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 = 3864 lbs (seismic) Vw = 8550 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL2	8,550	41.5	27.8	307.2	9.0	2,765	HDU2	SSTB20	16	(2) 2x6	2
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 = **139** plf < v_{allow} = 240 plf → OK (seismic)

v_w = **307** 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 = 10.75 ft
Max. AR: h/bs = **0.84** → 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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 9.0 ft
ρ = 1.0 (out-of-plane)

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

V_{sperp} = 614 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 4.5 ft

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

V_{wperp} = 4,279 lbs

Wind Load Governs:

V_{perp} = 4,279 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	4,279	9.2	**4.5**
Para. Load	8,550	10.1	**4.1**

La = 41.5 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 2.

A35 Framing Angle Spacing

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

Lac = 27.8 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 307.2 plf
Spacing = **2.0** ft

Use A35 clips for top plt./blocking connection @ 24" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 194.4 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.088 in (Simpson Holdown)
en = 0.0046 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	17.083	0.10	in
2	10.75	0.13	in
3	0	--	in
4	0	--	in
5	0	--	in
	Max. Defl.	**0.13**	in

Cd = 4
ASCE 7-10 Δ = 0.52 in
(Table 12.12-1) Δlimit = 2.16 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#: 2015-048

Bearing on Wall Plates

Top/Sill Plt. Species	HF	
$F_{c\perp}$	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.125)
$F_{c\perp}'$	405.00	psi
Ab	16.50	in^2
Pc	3239	lbs
$f_{c\perp}$	196	psi
CSI (bearing)	**0.48** →	**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	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	168	psi
CSI (tension)	**0.14** →	**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	1.10	
(le/d)x	18.82	
E'min	580,000	psi
FcE	1346	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.567	
1 + FcE/Fc*/2c	0.979	
Cp	0.479	
Fc'	1137	psi
fc	196	psi
CSI (compression)	**0.17** →	**OK**

Shearwall Gravity Loads

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

(plf)	WDL	WLL	WSL/WLrL
Wall Loads	356	0	329

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

Pw = 2,765 lbs

Ps = 1,249 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	3,239	plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	2,877	plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,724	plf
6b.) D + .75L + .75E + 75S =	1,741	plf

* SWL Chord Tension =	**2,765**	lbs
SWL Chord Comp. =	**3,239**	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 =	307.2	plf
Spacing =	**8.8**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

Sill Plate at Foundation

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

*Only applicable at first story shearwalls.

SHEAR WALL CALCULATOR **SWL1** Vs = 2852 lbs (seismic) Vw = 5769 lbs (wind) **Job#: 2015-048**

SWL Name	Shear (lbs)	Wall Length (ft)	SWL Length	Unit Shear (plf)	Wall Hgt. (ft)	Uplift (lbs)	Holdown	Anchor Bolt	Embedment	Studs	Panels
SWL1	5,769	56.0	28.2	204.8	9.0	1,843	HDU2	SSTB16	12	(2) 2x6	4
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 = **101** plf < v_{allow} = 227 plf ⟶ OK (seismic)

v_w = **205** 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 = 3.5 ft
Max. AR: h/bs = **2.57** ⟶ OK
Max. AR Seismic Reduction: 2bs/h = 0.78

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 = 0.979 g
Ie = 1.0
ka = 1.0 (concrete is rigid)
Wall Hgt. = 9.0 ft
ρ = 1.0 (out-of-plane)

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

V_{sperp} = 829 lbs

Out-of-Plane Wind (MWFRS)

Ww = 38.19 psf
Ltrib = 4.5 ft

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

V_{wperp} = 5,774 lbs

Wind Load Governs:

V_{perp} = 5,774 lbs

AB Spacing	V (lbs)	# of Bolts	Spacing (ft)
Perp. Load	5,774	12.4	**4.5**
Para. Load	5,769	6.8	**8.2**

La = 56.0 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 = 28.2 ft (available collector length)
Fallow = 600 lbs (F1 direction)
Unit Shear = 204.8 plf
Spacing = **2.9** ft

Use A35 clips for top plt./blocking connection @ 32" o/c spacing.

Deflection (based on strength-level seismic forces)

v_u = 141.8 plf
E = 1,600,000 psi
A = 16.5 in^2
Gt = 83,500 plf (Table C4.2.2A)
da = 0.088 in (Simpson Holdown)
en = 0.0018 in (Table C4.2.2D)
nail spacing = 6 in
Sht. both sides = NO

Panel #	b (ft)	Δs	
1	3.5	0.26	in
2	10.583	0.10	in
3	10.583	0.10	in
4	3.5	0.26	in
5	0	--	in
	Max. Defl.	**0.26**	in

Cd = 4
ASCE 7-10 Δ = 1.05 in
(Table 12.12-1) Δlimit = 2.16 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#: 2015-048

<u>Bearing on Wall Plates</u>

Top/Sill Plt. Species	HF	
$F_{c\perp}$	405	psi
Ct_c⊥	1.00	
CM_c⊥	1.00	
Cb	1.00	(1.125)
$F_{c\perp}'$	405.00	psi
Ab	16.50	in^2
Pc	2318	lbs
$f_{c\perp}$	140	psi
CSI (bearing)	**0.35**	→ **OK**

<u>Chord in Tension</u> (DF No. 2)

Ft	575	psi
CM_t	1.00	
Ct_t	1.00	
Ci_t	1.00	
CD	1.60	(wind)
CF_t	1.30	
Ft'	1196	psi
An	16.50	in^2
ft	112	psi
CSI (tension)	**0.09**	→ **OK**

<u>Chord in Compression</u> (DF No. 2)

Fc	1350	psi
CM_c	1.00	
Ct_c	1.00	
Ci_c	1.00	
CD	1.60	(wind)
CF_c	1.10	
(le/d)x	18.82	
E'min	580,000	psi
FcE	1346	psi
Fc*	2376	psi
c	0.80	sawn lumber
FcE/Fc*	0.567	
1 + FcE/Fc*/2c	0.979	
Cp	0.479	
Fc'	1137	psi
fc	140	psi
CSI (compression)	**0.12**	→ **OK**

<u>Shearwall Gravity Loads</u> (Point loads are assumed to bear directly above SWL chord)

(plf)	W_{DL}	W_{LL}	W_{SL}/W_{LrL}
Wall Loads	356	0	329

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

Pw =	1,843 lbs
Ps =	911 lbs

Wind ASD Load Cases from ASCE 7-10:

5.) D + W =	2,318 plf (governs)
6a.) D + .75L + .75W + 75(Lr or S) =	2,186 plf

Seismic ASD Load Cases from ASCE 7-10:

5.) D + E =	1,386 plf
6b.) D + .75L + .75E + 75S =	1,487 plf

* SWL Chord Tension =	**1,843** lbs
SWL Chord Comp. =	**2,318** lbs
Stud Spacing =	16 in
Chord Studs =	(2) 2x6
Chord Depth (dx) =	5.5 in
lb =	3.00 in

<u>Bottom Plate (Sole Plt.) Attachment to Floor</u>

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 =	204.8	plf
Spacing =	**13.2**	in

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

Nail 2x bottom plate to rim joist below w/ 16d nails @ 4" o/c spacing.

<u>Sill Plate at Foundation</u>

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

*Only applicable at first story shearwalls.

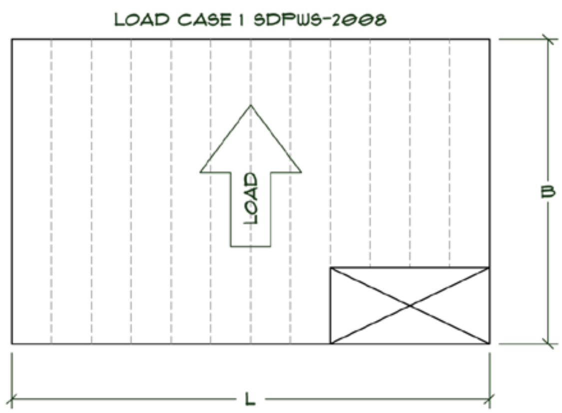
Roof Diaphragm and Sheathing Calculations

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 =	363	plf	L =	26	ft
Mmax =	30,674	ft-lbs	b =	31.5	ft
T=C=M/b=	**974**	**lbs**			
V =	4719	lbs			
v = V/b =	**150**	**plf**			

Load Case 1
SDPWS-2008



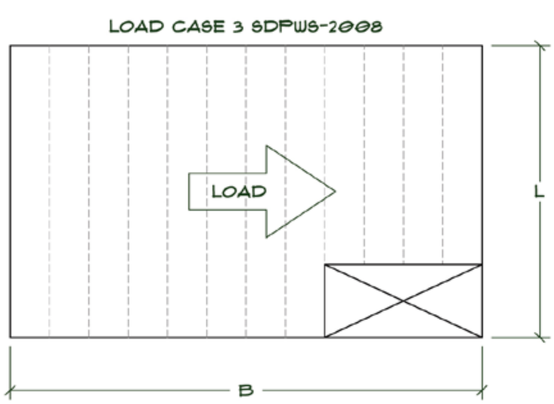
L/B Ratio = 0.83 → **OK**

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

2.) Longitudinal Load Case:

W_D =	321	plf	L =	31.5	ft
Mmax =	39,814	ft-lbs	b =	26	ft
T=C=M/b=	**1,531**	**lbs**			
V =	5056	lbs			
v = V/b =	**194**	**plf**			

Load Case 3
SDPWS-2008



L/B Ratio = 1.21 → **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:	15/32 in.	APA Rating:	Grade 32/16	
Nails:	8d		Sheathing Type:	PLY
Rafter/Truss Spacing:	24 in. o/c		Roof Framing Species:	SPF
(Unblocked Diaphragm)			SGAF:	0.92

Load Case 1: (transverse)

v = 150 plf < v_W = 308 plf ⟶ **OK**

Load Case 3: (longitudinal)

v = 194 plf < v_W = 232 plf ⟶ **OK**

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

Sheath roof with 15/32 APA rated PLY (Grade 32/16) 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 = **1,531** 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' = 6.8 nails

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

Use (14) - 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): 155.00 MPH

P_{3OH} = 145.79 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{3OH_ASD} : 87.474 psf

Roof Sheathing Nailing

	Edges (in.)	Field (in.)
Interior (Zone 1)	6	12
Perimeter (Zone 2)	4	4
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 = 7.8 psf + 38.5 psf = 46.3 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 = 87.5 psf < P_{allow} = 96.9 psf → **OK**

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

Sheathing Perpendicular to Rafters/Trusses

L/240	→	81 psf	>	61.2 psf [2]	→	**OK**
L/180	→	108 psf	>	61.2 psf [2]	→	**OK**
Bending	→	123 psf	>	87.5 psf	→	**OK**
Shear	→	285 psf	>	87.5 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 @ 4" o/c edges, 4" 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): 155.00 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 = 63.58 psf (unfactored)

Convert to ASD value by multiplying by 0.6:

P_{5_ASD} = 38.148 psf

Wall Sheathing Nailing

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

*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} = 38.1 psf < P_{allow} = 37.5 psf → **NG**

Sheathing Perpendicular to Studs

P_{5_ASD} = 38.1 psf < P_{allow} = 190.6 psf → **OK**

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

Sheathing Parallel to Studs

L/360	→	26 psf	>	26.7 psf [2]	→ **NG**
Bending	→	86 psf	>	38.1 psf	→ **OK**
Shear	→	331 psf	>	38.1 psf	→ **OK**

Sheathing Perpendicular to Studs

L/360	→	128 psf	>	26.7 psf [2]	→ **OK**
Bending	→	288 psf	>	38.1 psf	→ **OK**
Shear	→	331 psf	>	38.1 psf	→ **OK**

Sheath walls with 7/16 APA rated OSB (Grade 24/16) strength axis perpendicular to studs 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, 6" 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.

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

Show Map   GeoData Viewer

Parcel 090500054200
Situs Address 00164 OCTOPUS AVE NE
Legal Description DIV 3 LOT 542

Owner	BARTH GEORGE & LORI	**File Updated**	10/7/2015 10:05
Address	101 SKOOKUMCHUCK ST SE OCEAN SHORES, WA 98569	**Location**	T 17 R 12 Sec 02

Certified Values:	Land	Building	Combined
	$20,000.00	$1,000.00	$21,000.00

Year Built	0000	**Tax Code**	OS064 H2
Building Type		**School District**	064
Style		**Voting Precinct**	801 Ocean Shores
Quality		**Total Acres**	0
		Fire Patrol Acres	0

(pdf) **Land Use** 18 - All Other Residential Not Elsewhere Coded (Bare Land Platted & Outside Plats and Sheds in City Limits)

	Square Feet	Type
Lot	8400	
Building SF	0	
Percentage Complete	100%	
Basement SF	0	
Finished Basement SF	0	
Foundation		
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
8/18/2003	E162865	$24,500.00	WD	BL
6/10/2015	E216164	$30,000.00	WD	BL