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ENGINEERING REPORT

STRUCTURAL REVIEW

July 8, 2014

JOB NUMBER: **2014-002**

PLAN NUMBER: PORCH1917

CUSTOMER: CASEY A. DOYLE

LOCATION: 24 BENNETT LANE OAKVILLE, WA 98568

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 24 Bennett Lane Oakville, WA 98568.

Job#: 2014-002

ENGINEERING REPORT: STRUCTURAL REVIEW

Customer: Casey A. Doyle
Location: 24 Bennett Lane Oakville, WA 98568
Engr: Nathaniel P. Wilkerson
Date: 8-Jul-14

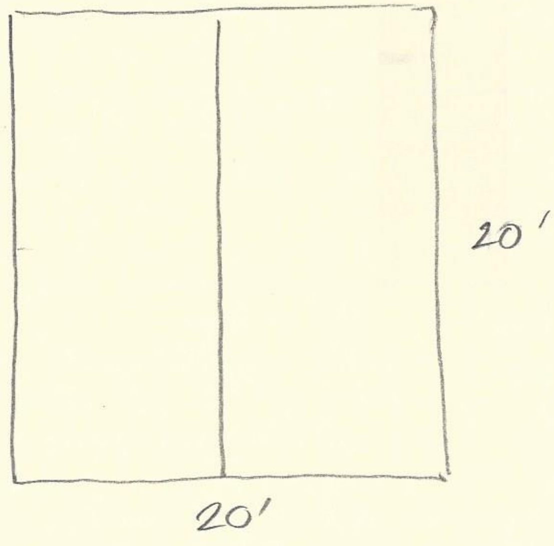
CODES

ICC International Building Code IBC2012
Minimum Design Loads for Buildings ASCE7-10
American Concrete Institute ACI 318-11
AWC NDS 2012

DESIGN CRITERIA SUMMARY

Wind Speed (ultimate)	130.0	MPH
Terrain Exp. Category	C	
Wind Risk Category	II	
Roof Live Load	20.0	PSF
Roof Dead Load	15.0	PSF
Seismic Design Category	D	
Site Class	D	Stiff Soil
Ground Snow Load	25.0	PSF
Frost Line Depth	12.0	IN
Occupancy Classification	R	
Risk Category	2.0	
Snow Importance Factor	1.0	PSF
Wind Factor in Load Combinations	0.6	
Seismic Importance Factor	1.0	
Construction Type	V-B	
Soil Bearing Capacity	1500.0	PSF
SDS	0.877	g
SD1	0.585	g
Roof Eve Height	9.000	FT
Peak Roof Height	12.333	FT
Mean Roof Height	10.667	FT
Latitude	46.8651	
Longitude	-123.2897	
Elevation:	70.0	FT

A.) Vertical Loads



Roof Plan

* disregard section framed over house assume those loads carried by main structure

[Gable Roof]

$A_{roof} = 400$ sqft.

From ASD Load Combinations:

3. D + S (governing load combination)

From Snow Load Calculator (per ASCE 7-10) balanced snow load is:

TOTAL LOAD VERTICAL = D + S = 15.8 psf + 21.0 psf = 36.8 psf

= 36.8 psf x roof area = 36.8 psf x 400 sqft

= 14,720 lbs

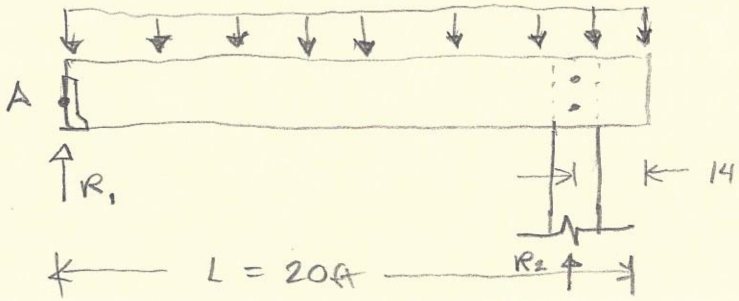
1) Check Glulam Beam

Assume each beam supports half of roof load (distributes).

$L_{beam} = \frac{14,720 \text{ lbs}}{2}$

= 7,360 lbs

$= \frac{7360 \text{ lbs}}{20 \text{ ft}} = 368$ plf



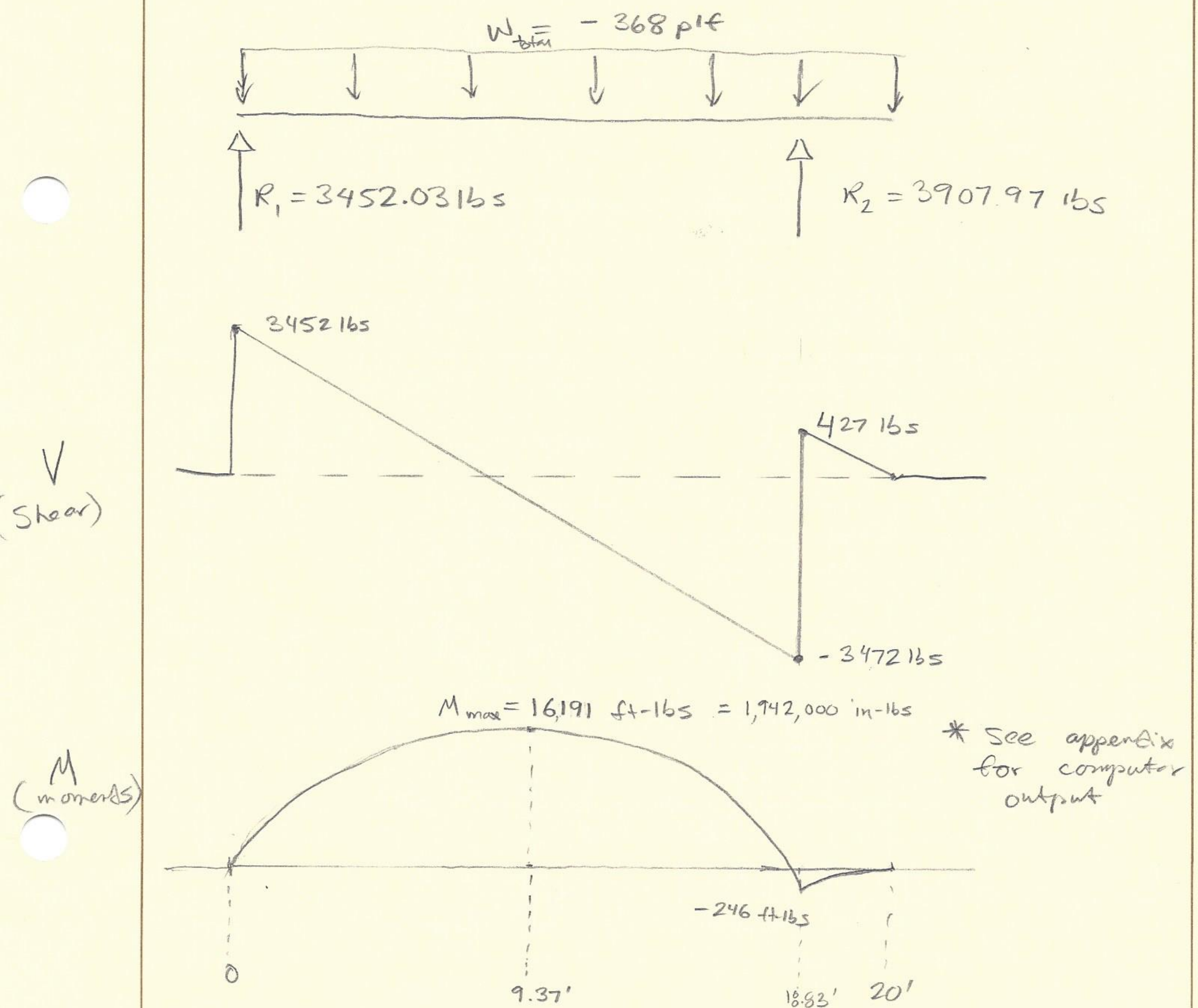
Summing Moments at point A:

$$\sum \vec{M}_A = -(7360\ \text{lbs})(10\ \text{ft}) + R_2(18.833\ \text{ft}) = 0$$

$$R_2 = \frac{(7360\ \text{lbs})(10\ \text{ft})}{(18.833\ \text{ft})} = 3907.97\ \text{lbs}$$ @ post

$$R_1 = 7{,}360\ \text{lbs} - 3907.97\ \text{lbs} = 3452.03\ \text{lbs}$$ @ beam hanger

Beam Diagrams



* See appendix for computer output

Given:

- Glulam Beam = 3 1/2 x 12 24F-V4 DF/DF
- Compression Edge of beam is braced with rafters to prevent lateral buckling
- Section Properties per APA (Form No. EWS S475H)

Beam Weight = 10.2 lb/ft

Area = 42 in2	F_b = 2,400 psi
S = 84 in3	E = 1,800,000 psi
I = 504 in4	F_v = 265 psi
EI = 907.2 x 10^6 lb-in2	C_D = 1.15 (snow loads)

Moment Cap. = 16,800 ft-lb

Shear Cap. = 7,420 lbs

then:

$$C_v = \left(\frac{21}{L}\right)^{.1} \times \left(\frac{12}{d}\right)^{.1} \times \left(\frac{5.125}{b}\right)^{.1} = \left(\frac{21}{18.83}\right)^{.1} \times \left(\frac{12}{12}\right)^{.1} \times \left(\frac{5.125}{3.5}\right)^{.1}$$

$C_v = 1.05$ ∴ $C_v = 1.0$

i) $F_b' = F_b(C_D)(C_M)(C_t)(C_v)$

$= (2400\text{ psi})(1.15)(1)(1)(1.0)$

$= 2760\text{ psi}$

$$f_b = \frac{M}{S} * \left(\frac{W_{total} + W_{beam}}{W_{total}}\right) = \frac{1{,}942{,}000\text{ in-lbs}}{84\text{ in}^3} * \left[\frac{368 + 10.2}{368}\right]$$

↑ to account for self weight of glulam beam

$f_b = 2377\text{ psi}$

$f_b = 2377\text{ psi} < F_b' = 2760\text{ psi}$ ⟹ OK

ii) $F'_{vx} = F_v (C_D)(C_M)(C_t)$

$= (265\,psi)(1.15)(1)(1)$

$= 304.8\,psi$

Ignore reduction of shear given by V* (conservative):

$$f_v = \frac{1.5\,V}{A} = \frac{(1.5)(3472\,lbs)}{42\,in^2}$$

$f_v = 124\,psi$

$f_v = 124\,psi < F'_{vx} = 304.8\,psi \Longrightarrow$ OK

iii)
For deflection assume (conservative) simply supported beam:

$E'_x = E_x (C_M)(C_t) = 1{,}800{,}000\,(1)(1) = 1{,}800\,000\,psi$

$$\Delta TL = \frac{5(W_{total} + W_{beam})\,L^4}{384\,E'\,I}$$

$$\Delta_{TL} = \frac{(5)(368\,plf + 10.2\,plf)(18.833\,ft)^4}{(384)(1800\,000\,\frac{lbs}{in^2})(504\,in^4)} \times \left(\frac{12\,in}{ft}\right)^3$$

$\Delta_{TL} = 1.18\,in$

$$(L/d)_{TL} = \frac{L}{\Delta_{TL}} = \frac{18.833\,ft}{1.18\,in} \times \left(\frac{12\,in}{1\,ft}\right) = 191$$

$$\frac{L}{191} < \frac{L}{180} \Longrightarrow \text{OK}$$

iv) By proportion:

$$\Delta_{LL} = \frac{S}{W_{total} + W_{beam}} \Delta_{TL} = \left(\frac{210\ plf}{368\ plf + 16.2\ plf}\right) 1.18\ in = .65\ in$$

$$\left(L/d\right)_{LL} = \frac{L}{\Delta_{LL}} = \frac{8.333\ ft}{.65\ in} \times \left(\frac{12\ in}{1\ ft}\right) = 345$$

$$\frac{L}{345} < \frac{L}{240} \Longrightarrow OK$$

v) Bearing: (Check Bearing at Beam Hanger)

From NDS Table 5A $F_{c\perp} = 650\ psi$

$$F'_{c\perp} = 650\ psi\ (C_M)(C_t)(C_b)$$
$$= 650\ psi\ (1)(1)(1)$$
$$= 650\ psi$$

$$R = R_1 + \left(\frac{W_{beam}\ L_{beam}}{2}\right) = 3452\ lbs + \frac{(10.2\ plf)(18.83\ ft)}{2}$$

$$R = 3548\ lbs$$

$$Req.\ A = \frac{R}{F'_{c\perp}} = \frac{3548\ lbs}{650\ psi} = 5.45\ in^2$$

$$A_{hanger} = 2.5\ in \times 3.5\ in = 8.75\ in^2$$

$$A_{hanger} = 8.75\ in^2 > Req.\ A = 5.45\ in^2 \Longrightarrow OK$$

2) Check 6×6 Post

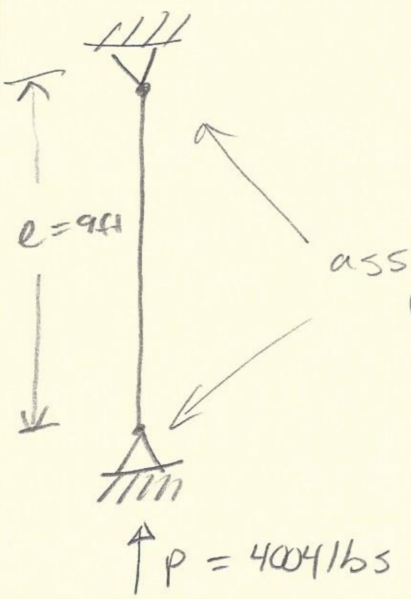
Load on post is R_2 and half the weight of Glulam beam:

$$P = R_2 + \frac{W_{beam} L_{beam}}{2}$$

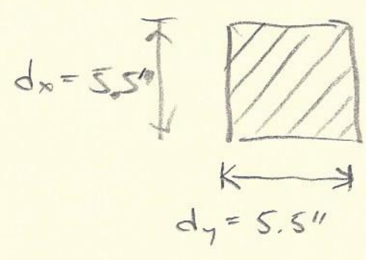
$$= 3907.9\text{ lbs} + \frac{(10.2\text{ plf})(18.83\text{ ft})}{2}$$

$$= \underline{4004\text{ lbs}}$$

Free Body Diagram of Post:



$C_F = 1.0$ for compression of posts & timbers



$A = 30.25\text{ in}^2$

Values from NDS Table 4D:

Assume Post is DF No. 1

$F_c = 1000\text{ psi}$

$E_{min} = 580{,}000\text{ psi}$

$$\left(\frac{l_e}{d}\right)_{max} = \frac{1.0\,(9\text{ ft} \times 12\frac{\text{in}}{\text{ft}})}{5.5\text{ in}} = 19.64$$

$$E'_{min} = E_{min}\, C_M C_t C_i$$

$$= 580{,}000\,psi\,(1)(1)(1)$$

$$= 580{,}000\,psi$$

c = 0.8 for sawn lumber columns

$$F_{cE} = \frac{.822\,E'_{min}}{(l_e/d)^2} = \frac{.822\ 580{,}000\,psi}{(19.64)^2} = 1236.4\,psi$$

$$F_c^* = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)$$

$$= 1000\,psi\,(1.15)(1)(1)(1)(1) = 1150\,psi$$

$$\frac{F_{cE}}{F_c^*} = \frac{1236.4\,psi}{1000\,psi} = 1.075$$

$$\frac{1 + F_{cE}/F_c^*}{2c} = \frac{1 + 1.236}{2(.8)} = 1.297$$

$$C_p = \frac{1 + F_{cE}/F_c^*}{2c} - \sqrt{\left[\frac{1 + F_{cE}/F_c^*}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

$$= 1.297 - \sqrt{(1.297)^2 - \frac{1.075}{.8}}$$

$$= .715$$

$$F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_p)(C_i)$$

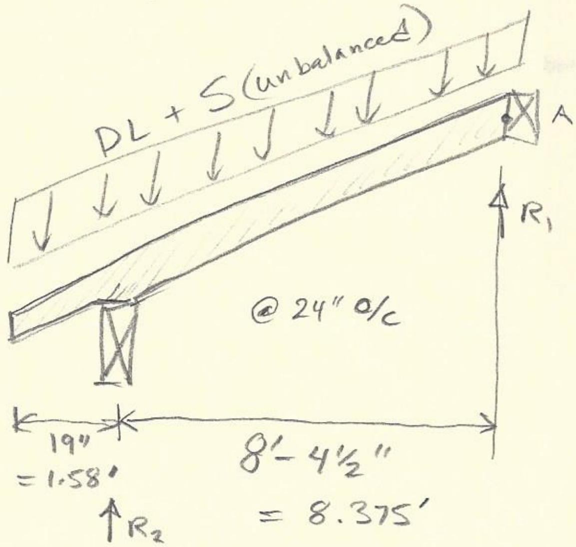
$$= 1000\,psi\,(1.15)(1)(1)(1)(.715)(1)$$

$$F_c' = 822\,psi$$

$$f_c = \frac{P}{A_g} = \frac{4004\,lbs}{30.25\,in^2} = 132.4\,psi$$

$f_c = 132.4\,psi < F_c' = 822\,psi \Longrightarrow$ OK

3) Check 2x6 Rafters



From snow load calculator governing snow load is unbalanced load on leeward side

$$TL = D + S_{(unbalanced)}$$
$$= 15.8\,psf + 25\,psf$$
$$= 40.8\,psf$$

tributary area = $2' \times 8.375' + (2' \times 1.58')$
= $19.91\,ft^2$

$$TL = (40.8\,psf)(19.91\,ft^2) = 812.3\,lbs$$

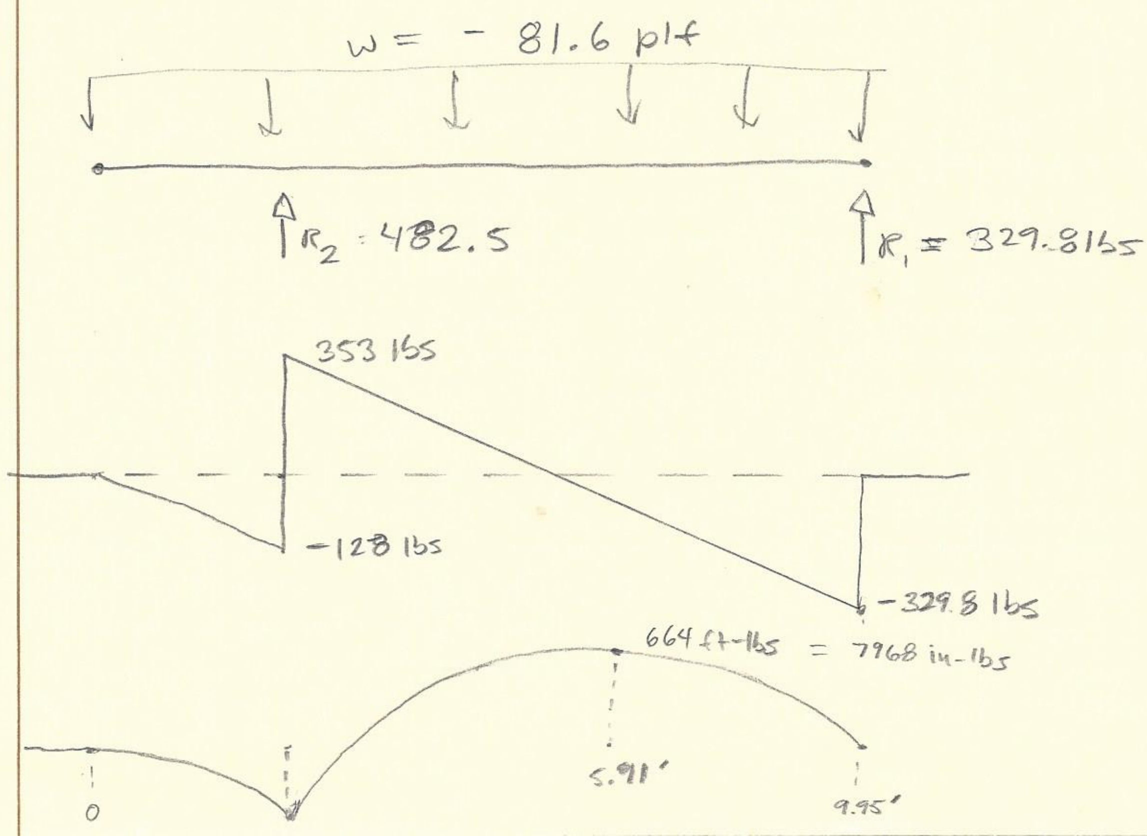
Summing Moments at point A:

$$\Sigma \vec{M}_A = 812.3\,lbs\left(\frac{9.95\,ft}{2}\right) - R_2(8.375) = 0$$

$$R_2 = 482.5\,lbs$$

$$R_1 = 812.3\,lbs - 482.5\,lbs = 329.8\,lbs$$

BEAM DIAGRAMS



Given: 2x6 Rafter | HF No. 2 |

$A = 8.25 \text{ in}^2$ From NDS Table 4A:

$S = 7.56 \text{ in}^3$ $E = 1300000$

$F_b = 850 \text{ psi}$ $C_F = 1.3$

$F_v = 150 \text{ psi}$ $C_R = 1.15$ (repetitive member increase)

$F_{c\perp} = 405 \text{ psi}$ $C_D = 1.15$ (Snow)

i) $F_b' = F_b C_D C_M C_t C_L C_F C_R C_i$ $I = 20.79 \text{ in}^4$

$= 850 (1.15)(1)(1)(1)(1.3)(1.15)(1)$

$= 1461.4 \text{ psi}$

$$f_b = \frac{M}{S} = \frac{7968 \text{ in-lbs}}{7.56 \text{ in}^3} = 1053.9 \text{ psi}$$

$f_b = 1053.9 \text{ psi} < F_b' = 1461.4 \text{ psi} \Longrightarrow$ OK

ii) Ignore shear reduction given by V^*:

$F_v' = F_v C_D C_M C_t C_i$

$= 150 \text{psi} (1.15)(1)(1)(1)$

$= 172.5 \text{ psi}$

$$f_v = \frac{1.5V}{A} = \frac{1.5 (353 \text{ lbs})}{8.25 \text{ in}^2} = 64.2 \text{ psi}$$

$f_v = 64.2 \text{ psi} < 172.5 \text{ psi} = F_v' \Longrightarrow$ OK

iii) For deflection assume simply supported beam between bearing points (conservative)

$E' = E' C_M C_t C_i$

$= 1300000 \text{ psi} (1)(1)(1)$

$= 1300000$

$$\Delta_{TL} = \frac{5\, wL^4}{384\, EI}$$

$$= \frac{5\,(81.6\ \text{lb/ft})\,(8.375\ \text{ft})^4}{384\,(1300\,000)\,(20.79\ \text{in}^4)} \times \left(12\ \frac{\text{in}}{\text{ft}}\right)^3$$

$$\Delta_{TL} = .334\ \text{in}$$

$$\left(L/d\right)_{TL} = \frac{L}{\Delta_{TL}} = \frac{8.375\ \text{in}}{.334\ \text{in}} \times \left(\frac{12\ \text{in}}{\text{ft}}\right) = 300$$

$$L/300 < L/180 \Longrightarrow \text{OK}$$

iv) By proportion:

$$\Delta_{LL} = \frac{S}{W_{total}}\,\Delta_{TL} = \left(\frac{50}{81.6}\right).334 = .204\ \text{in}$$

$$\left(L/d\right)_{LL} = \frac{L}{\Delta_{LL}} = \frac{8.375\ \text{in}}{.204\ \text{in}} \times \left(\frac{12\ \text{in}}{\text{ft}}\right) = 491$$

$$L/491 < L/240 \Longrightarrow \text{OK}$$

v) Bearing Check (@ birds mouth cut on glulam)

$$F'_{c\perp} = F_{c\perp}\, C_M C_t\, C_b = 405\ \text{psi}\,(1)(1)(1)$$

$$F'_{c\perp} = 405\ \text{psi}$$

$$R = R_2 = 482.5\ \text{lbs}$$

$$\text{Req } A = \frac{R}{F'_{c\perp}} = \frac{482.5\ \text{lbs}}{405\ \text{psi}} = 1.19\ \text{in}^2$$

$$A_{cut} = 1.5\ \text{in} \times 3.5\ \text{in} = 5.25\ \text{in}^2$$

$$A_{cut} = 5.25\ \text{in}^2 > \text{Req } A = 1.19\ \text{in}^2 \Longrightarrow \text{OK}$$

B. Lateral Loads

Basic Wind Speed = 130 mph

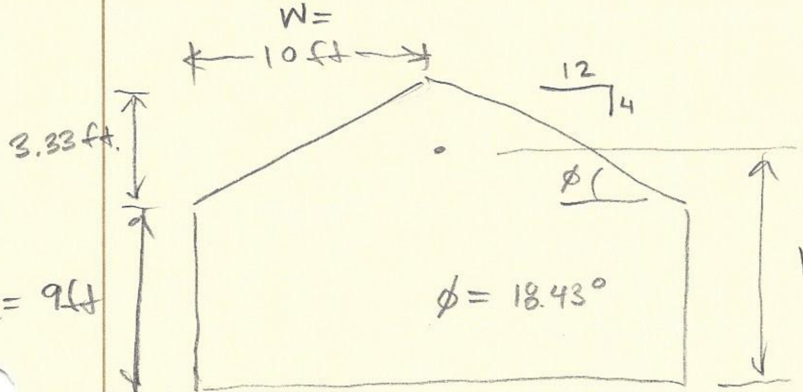
Structure qualifies as an open building with three open walls

Wind Risk Cat. = 2

Exposure = C

Eave Height ≈ 9 ft.

GC_{pi} = 0.0 from Table 26.11-1 ASCE 7-10



h = mean roof height = $\frac{W \tan\phi}{2} + h_e$

$h = \frac{10 ft \tan 18.43}{2} + 9 ft$

h = 10.67 ft

For MWFRS:

$q_h = .00256\ K_h K_{zt} K_D V^2$

where: K_{zt} = 1.0 for homogeneous terrain

K_h = .85 (table 27.3-1 ASCE 7-10)

V = 130 mph

K_D = .85 for buildings

$q_h = .00256 (.85)(1.0)(.85)(130)^2$

q_h = 31.25 psf

for open buildings Sec. 27.4.3 from ASCE 7-10

$p = q_h G C_N$

where: $G = .85$ (gust effect factor)

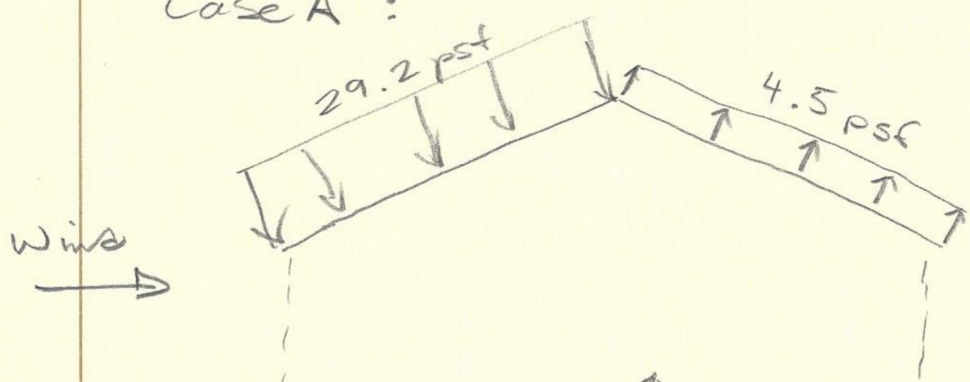
q_h = Velocity pressure at mean roof height

$p = 31.25 \text{ psf} \times .85 \times C_N$

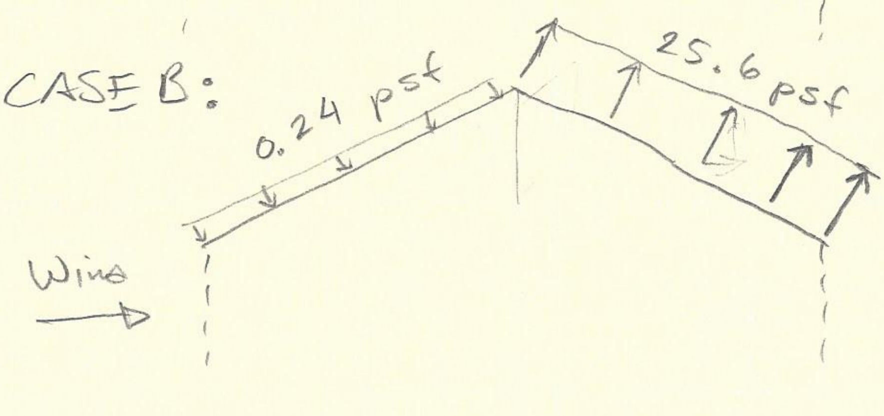
linear interpolating values from Fig. 27.4-5:
and considering $\gamma = 0°$ as governing load case:

Roof Angle ϕ	LC	C_{NW}	C_{NL}
$\phi = 18.43°$	A	1.1	-.171
	B	.009	-.963

Case A:



CASE B:



then maximum lateral force is given by Case A:

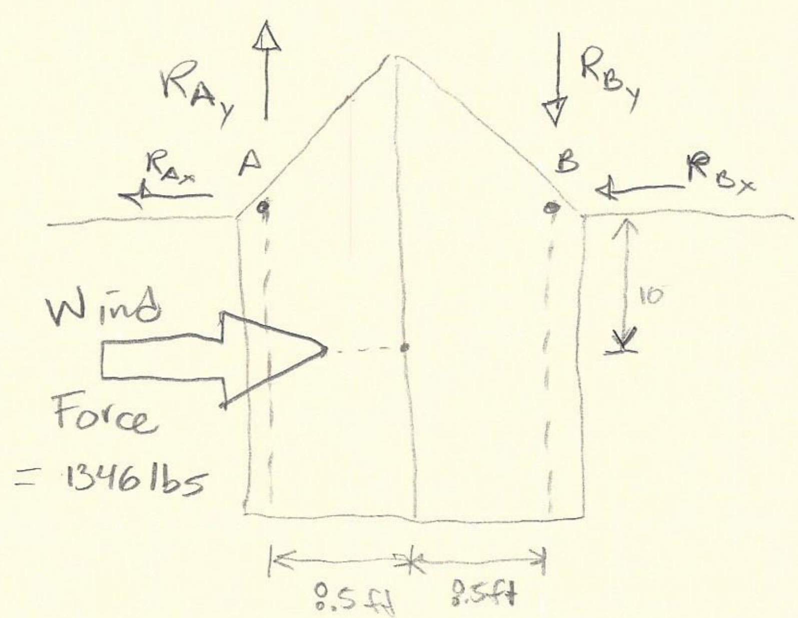
$$\text{Shear} = (29.2 + 4.5)_{psf}(20ft \times 3.33ft)$$
$$= 2244.42 \text{ lbs}$$

For ASD use .6W :

$$\text{Shear}_{ASD} = (2244.42 \text{lbs})(.6) = 1346 \text{ lbs}$$

Assume that wind force acts on centroid of roof surface and posts do not contribute any support for racking. Then twisting of structure is prevented by glulam beams anchored to walls (neglect contributions by roof sheathing).

"FBD of Shear Resistance"



Summing moments about A:

$$\Sigma \vec{M}_A = F_W \, 10_{ft} - R_{By}(8.5ft)(2) = 0$$

$$R_{By} = \frac{(1346\,lbs)(10\,ft)}{(8.5\,ft)(2)} = 792.1\,lbs$$

$$R_{Ay} = -R_{By} = -792.1\,lbs$$

Assume $R_{AX} = R_{BX}$ from symetry of problem

$\therefore \quad R_{AX} + R_{BX} = F_W$

$$R_{AX} = \frac{F_W}{2} = 673\,lbs$$

1) Resultant uplift force on SST HUC412TFX BEAM HANGER

$$R_{hanger} = \sqrt{R_{AX}^2 + R_{AY}^2}$$

$$= \sqrt{(792.1)^2 + (673)^2}$$

$$R_{hanger} = 1039\,lbs$$

compared with allowable loads for wind uplift

$$R_{hanger} = 1039\,lbs < 1125\,lbs \implies OK$$

2) Checking snow load on beam hanger:
from page 5 → $R = 3548\,lbs$

$$R = 3548\,lbs < 4885\,lbs \implies OK$$

3) Check Uplift on Rafters:

the max uplift is given on leeward side @ CASE B

We conservatively ignore dead load weight of asphalt shingles, sheathing (OSB) and 2x6 rafter then:

$$F_{uplift} = (A_{trib} \times 25.6\,psf)\,.6$$

where: $$A_{trib} = \frac{W}{\cos\phi}(2ft) = \frac{(10ft)(2ft)}{\cos 18.43} = 21.08\,ft^2$$

$$F_{uplift} = 323.8\ lbs$$

$$\Sigma \vec{M}_A = 323.8\,lbs\left(\frac{10ft}{2}\right) - U_2(8.375) = 0$$

$U_2 = 193.3$ lbs

$U_1 = 323.8\,lbs - 193.3\,lb = 130$ lbs

For LSU 26 Hanger:

$U_1 = 130\,lbs < 415\,lbs \Longrightarrow$ OK

Checking snow load for LSU26 hanger:

$R_1 = 329.8\,lb < 695\,lbs \Longrightarrow$ OK

For H1 Tie: (uplift only)

$U_2 = 193.3\,lbs < 400\,lbs \Longrightarrow$ OK

C.) Concrete and Post Anchor

1) Check bearing pressure, assume Soil bearing capacity of 1500 psf

from page 6:

$P = 4004 \text{ lbs}$ (snow load governs)

$$\text{Req. Soil Bearing} = \frac{P}{A_{footing}}$$

$A_{footing} = (2ft)(2ft)$

$$\text{Req Soil Bearing} = \frac{4004 \text{ lbs}}{4 \text{ ft}^2} \qquad A_{footing} = 4 \text{ ft}^2$$

$\text{Req. Soil Bearing} = 1001 \text{ psf} < 1500 \text{ psf} \Rightarrow OK$

2) Check ABU66 capacity:

$P = 4004 \text{ lbs} < 12{,}000 \text{ lbs} \longrightarrow OK$

3) Check ABU 66 uplift:

from wind calculations the max. uplift is given by CASE B on leewordside.

Assume post on leeward side takes half the leeward side uplift (conservative).

$P_{uplift} = -(.6D) + .6W$

where $D = \left(\frac{15.8}{36.8}\right) R_2 = \left(\frac{15.8}{36.8}\right) 3907.9 \text{ lbs} = 1677.8 \text{ lbs}$

$W = (25.6 \text{ psf})(10 ft)(10 ft) = 2560 \text{ lbs}$

$P_{uplift} = -.6(1677.8) + .6(2560) = \underline{529 \text{ lbs}}$

$P_{uplift} = 529 \text{ lbs} < 2300 \text{ lbs} \Longrightarrow OK$

Snow Load Report

1. Roof and Building Data

Ground Snow Load (Pg):	25.0 psf
Roof Pitch:	4 /12
Risk Category:	II
Eave-to-Ridge (W):	10 ft.
Terrain Category:	C
Exposure:	Partially Exposed
Thermal Factor (C_t):	1.20
Roof Surface:	Asphalt Shingles
Roof System:	rafter
Spacing:	24 in. o/c
Overhang:	16 in.

2. Design Loads

Top Chord Dead Load: 15 psf

Bottom Chord Dead Load: 0 psf

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

Adj. TCDL (TCDL x SF): 15.8 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:

$p_f = 0.7 C_e C_t I_s p_g$

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 = 1.20 = 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

$p_f = 0.7 C_e C_t I_s p_g = 0.7(1.00)(1.20)(1.00)(25.0) = 21.0$ 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 (18.43°) 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 = 1.20 the roof slope factor C_s is given by the solid line of ASCE 7-10 Figure 7-2c.

$p_s = C_s p_f = (1.00)(21.0) = 21.0$ psf

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

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

where:

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

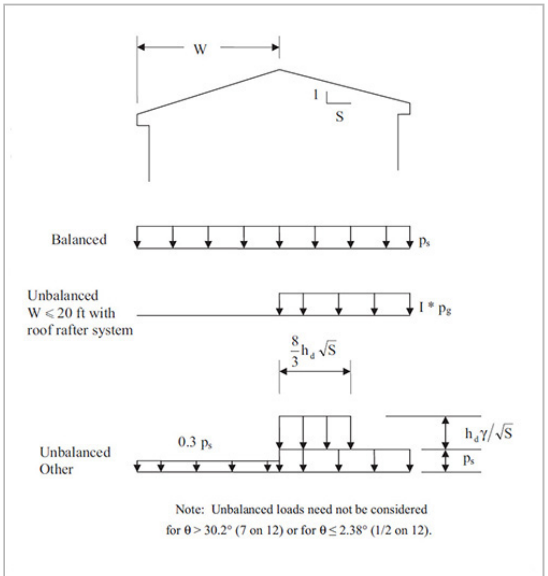


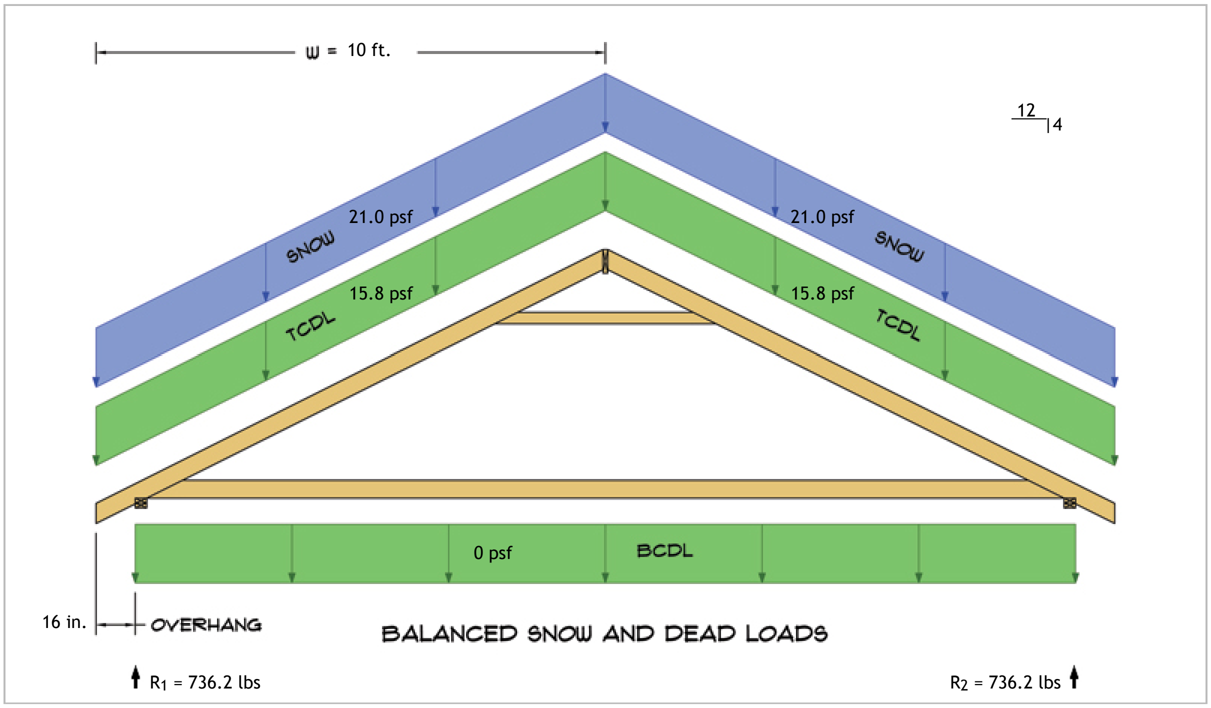
FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

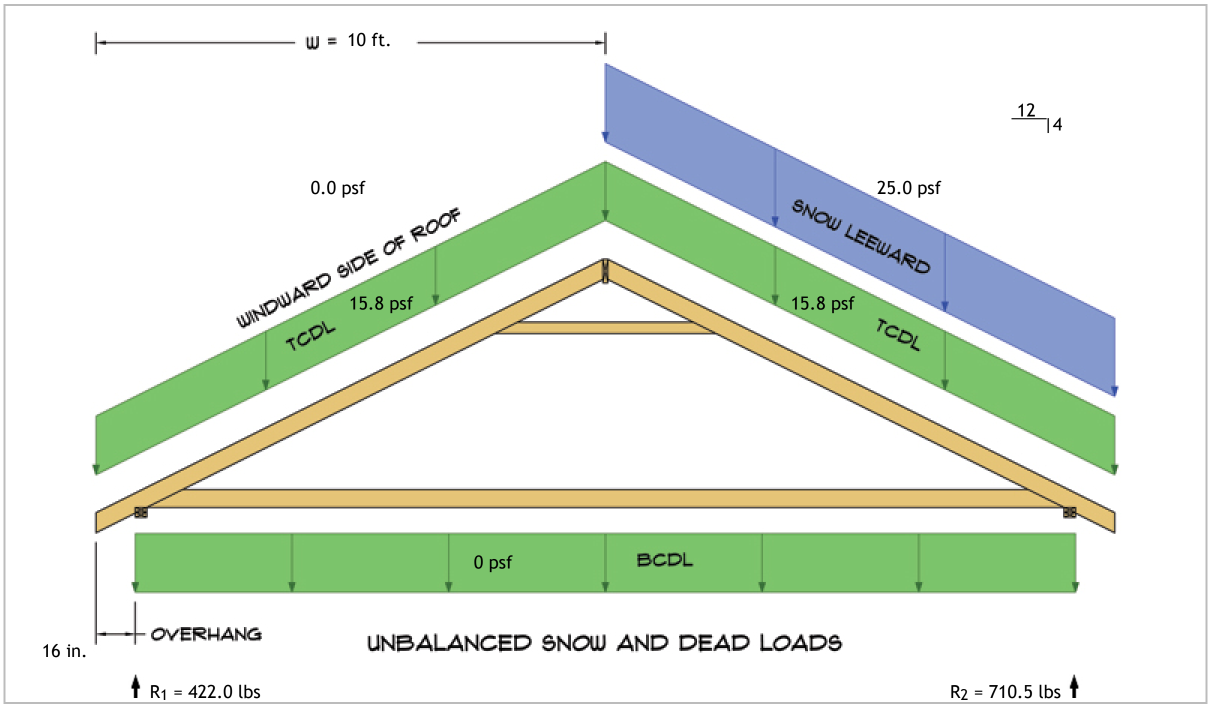
For a roof rafter system with W ≤ 20 ft., the simplified unbalanced snow load is given by the third diagram of ASCE Figure 7-5.

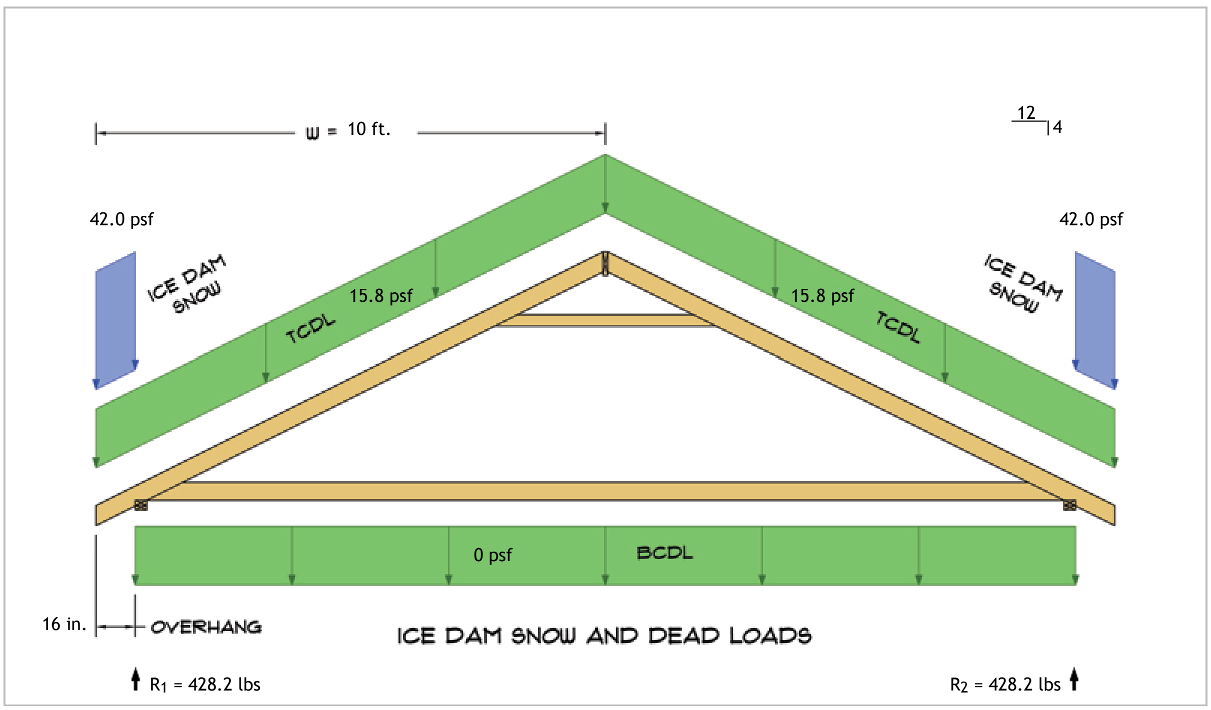
$p_{windward}$ = 0.0 psf
$p_{leeward} = I_s p_g$ = (1.00)(25.0) = 25.0 psf

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)(21.0) = 42.0 psf







*Disclaimer: The calculations produced herein are for initial design and estimating purposes only. The calculations and drawings presented do not constitute a fully engineered design. All of the load cases required to fully design an actual structure are not provided by this calculator. For the design of an actual structure, a registered and licensed professional should be consulted as per IRC 2012 Sec. R802.10.2 and designed according to the minimum requirements of ASCE 7-10. The snow load calculations provided by this online tool are for educational and illustrative purposes only. Medeek Design assumes no liability or loss for any designs presented and does not guarantee fitness for use.

Subject	Customer	Location	Job No.
Snow Loads	Casey Doyle	24 Bennet Lane Oakville, WA 98568	2014A53

Engr.: N. Wilkerson
Date: 7/7/2014

MEDEEK ENGINEERING INC.
3050 State Route 109 Copalis Beach, WA 98535
ph. (425) 741-5555 www.medeek.com

Rev.: -

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 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: City: State: Zip: Find

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

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

Design Maps Summary Report

User–Specified Input

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

Site Coordinates 46.8651°N, 123.2897°W

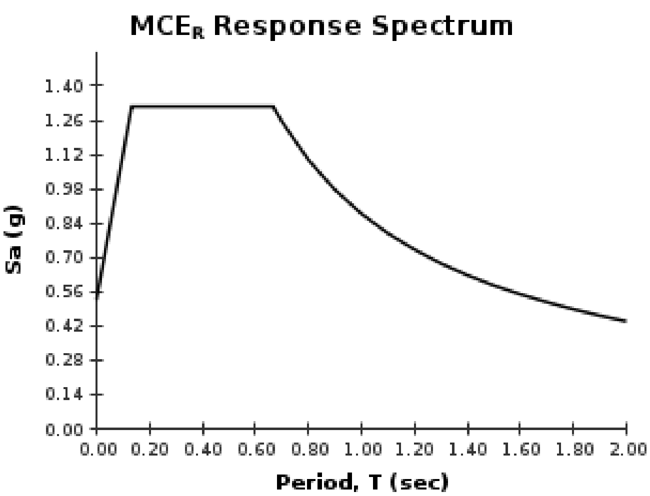
Site Soil Classification Site Class D – "Stiff Soil"

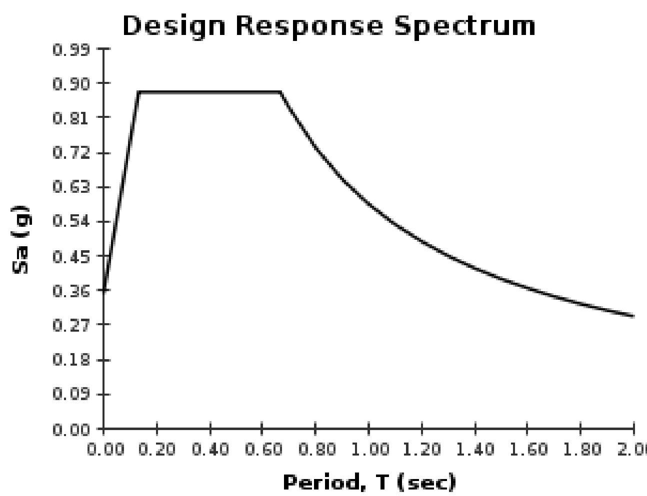
Risk Category I/II/III

USGS–Provided Output

S_S =	1.316 g	S_{MS} =	1.316 g	S_{DS} =	0.877 g
S_1 =	0.585 g	S_{M1} =	0.878 g	S_{D1} =	0.585 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.





Some loads in the table below have changed since this catalog was published. For updated loads, visit www.strongtie.com.

SIMPSON Strong-Tie®

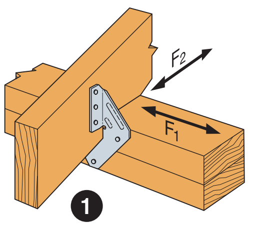
H/TSP Seismic & Hurricane Ties

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

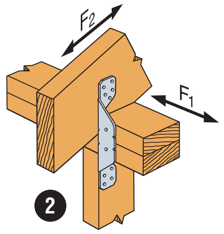
These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 27 for more information.

Model No.	Ga	Fasteners			DF/SP Allowable Loads			Uplift with 8dx1½ Nails (160)	SPF/HF Allowable Loads			Uplift with 8dx1½ Nails (160)	Code Ref.
		To Rafters/ Truss	To Plates	To Studs	Uplift (160)	Lateral (160) F1	Lateral (160) F2		Uplift (160)	Lateral (160) F1	Lateral (160) F2		
H1	18	6-8dx1½	4-8d	—	585	485	165	455	400	415	140	370	I17, L6, F16
H2A	18	5-8dx1½	2-8dx1½	5-8dx1½	575	130	55	—	495	130	55	—	IP1, L18, F25
H2ASS	18	5-SS8D	2-SS8D	5-SS8D	400	130	55	400	345	130	55	345	170
H2.5A	18	5-8d	5-8d	—	600	110	110	575	535	110	110	495	I17, F16
H2.5ASS	18	5-SS8d	5-SS8d	—	440	75	70	365	380	75	70	310	170
H2.5T	18	5-8d	5-8d	—	545	135	145	425	545	135	145	425	IP1, L18, F25
H3	18	4-8d	4-8d	—	455	125	160	415	320	105	140	290	I17, L6, F16
H4	20	4-8d	4-8d	—	360	165	160	360	235	140	135	235	
H5	18	4-8d	4-8d	—	455	115	200	455	265	100	170	265	
H6	16	—	8-8d	8-8d	950	—	—	—	820	—	—	—	I17, F16
H7Z	16	4-8d	2-8dx1½	8-8d	985	400	—	—	845	345	—	—	
H8	18	5-10dx1½	5-10dx1½	—	745	75	—	630	565	75	—	510	L10, F26
H10A Sloped	18	9-10dx1½	9-10dx1½	—	855	590	285	—	760	505	285	—	
H10A	18	9-10dx1½	9-10dx1½	—	1140[7]	590	285	—	1015	505	285	—	I17, L18, F25
H10ASS	18	9-SSN10	9-SSN10	—	970	565	170	—	835	485	170	—	170
H10AR	18	9-10dx1½	9-10dx1½	—	1050	490	285	—	905	420	285	—	
H10S[9,10]	18	8-8dx1½	8-8dx1½[10]	8-8d	1010	660	215	550	870	570	185	475	IP1, L18, F25
H10A-2	18	9-10dx1½	9-10dx1½	—	1245	815	260	—	1070	700	225	—	F25
H10-2	18	6-10d	6-10d	—	760	455	395	—	655	390	340	—	I17, F16
H11Z	18	6-16dx2½	6-16dx2½	—	830	525	760	—	715	450	655	—	170
H14	18	1 12-8dx1½	13-8d	—	1350[7]	515	265	—	1050	480	245	—	IP1, L18, F25
		2 12-8dx1½	15-8d	—	1350[7]	515	265	—	1050	480	245	—	
TSP	16	9-10dx1½	6-10dx1½	—	740	310	190	—	635	265	160	—	F26
		9-10dx1½	6-10d	—	890	310	190	—	765	265	160	—	

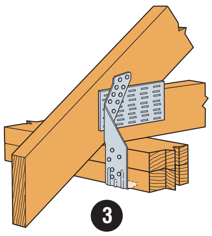
1. Loads have been increased for wind or earthquake loading with no further increase allowed: reduce where other loads govern.
2. Allowable loads are for one anchor. A minimum rafter thickness of 2½" must be used when framing anchors are used on each side of the joist and on the same side of the plate *(exeption: connectors installed such that nails on opposite side don't interfere)*.
3. Allowable DF/SP uplift load for stud to bottom plate installation *(see detail 15)* is 390 lbs. (H2.5A); 265 lbs. (H2.5ASS); 360 lbs. (H4) and 310 lbs. (H8). For SPF/HF values multiply these values by 0.86.
4. Allowable loads in the F_1 direction are not intended to replace diaphragm boundry members or cross grain bending of the truss or rafter members.
5. When cross-grain bending or cross-grain tension cannot be avoided in the members, mechanical reinforcement to resist such forces may be considered.
6. Hurricane Ties are shown on the outside of the wall for clarity and assume a minimum overhang of 3½". Installation on the inside of the wall is acceptable *(see General Instructions for the Installer notes u on page 17)*. For uplift Continuous Load Path, connections in the same area *(i.e. truss to plate connector and plate to stud connector)* must be on the same side of the wall.
7. Southern Pine allowable uplift loads for H10A = 1340 lbs. and for the H14 = 1465 lbs.
8. Refer to Simpson Strong-Tie® technical bulletin T-HTIEBEARING for allowable bearing enhancement loads.
9. H10S can have the stud offset a maximum of 1" from rafter *(center to center)* for a reduced uplift of 890 lbs. (DF/SP) and 765 lbs. (SPF).
10. H10S nails to plates are optional for uplift but required for lateral loads.
11. Some load values for the stainless-steel connectors shown here are lower than those for the carbon-steel versions. Ongoing test programs have shown this to also be the case with other stainless-steel connectors in the product line that are installed with nails. Visit ***www.strongtie.com/corrosion*** for updated information.
12. **NAILS:** 16dx2½ = 0.162" dia. x 2½" long, 10d = 0.148" dia. x 3" long, 10dx1½ = 0.148" dia. x 1½" long, 8d = 0.131" dia. x 2½" long, 8dx1½ = 0.131" dia. x 1½" long. See page 22-23 for other nail sizes and information.
13. **SCREWS:** Strong-Drive® SD #9x1½" (model SD9112) = 0.131" dia. x 1½" long *(for the models marked with the orange flag only)*. Full table loads apply.

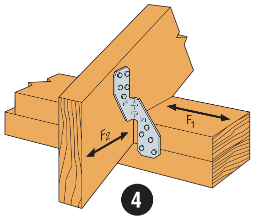


1 **H1 Installation**

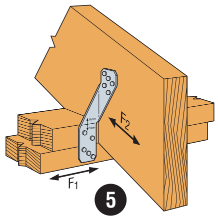


2 **H2A Installation**

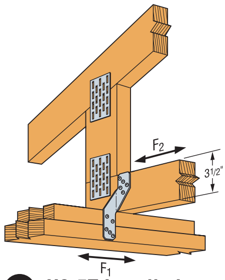

3 **TSP Installation**



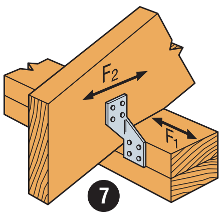
4 **H2.5A Installation**
(Nails into both top plates)



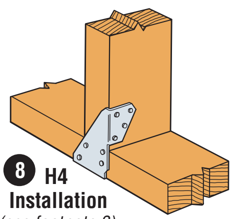
5 **H2.5T Installation**
(Nails into both top plates)

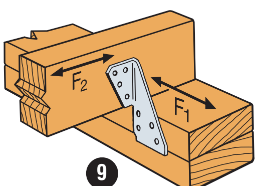


6 **H2.5T Installation**

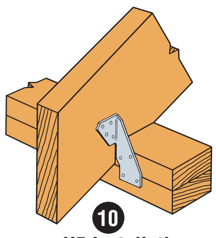


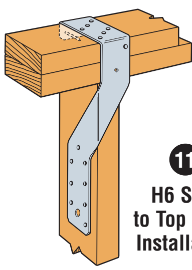
7 **H3 Installation**
(Nails into upper top plate)

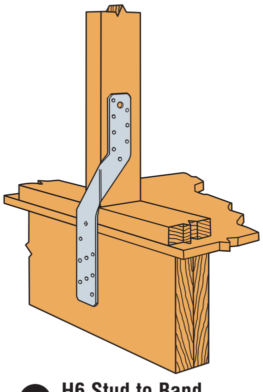

8 **H4 Installation**
(see footnote 3)

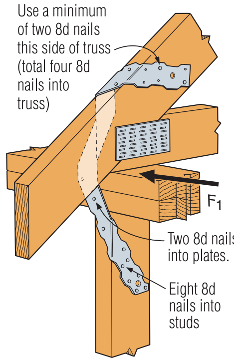


9 **H4 Installation**
(Nails into upper top plate)


10 **H5 Installation**
(Nails into both top plates)


11 **H6 Stud to Top Plate Installation**


12 **H6 Stud to Band Joist Installation**



13 **H7Z Installation**

Straps & Ties

ABA/ABU/ABW *Adjustable and Standoff Post Bases*

SIMPSON Strong-Tie®

Additional standoff bases are on page 214.

The AB series of retrofit adjustable post bases provide a 1" standoff for the post, are slotted for adjustability and can be installed with nails, Strong-Drive® SD screws or bolts (ABU). Depending on the application needs, these adjustable standoff post bases are designed for versatility, cost-effectiveness and maximum uplift performance.

Features:

- The slot in the base enables flexible positioning around the anchor bolt, making precise post placement easier
- The 1" standoff helps prevent rot at the end of the post and meets code requirements for structural posts installed in basements or exposed to weather or water splash

MATERIAL: Varies (see table)

FINISH: All galvanized, most offered in ZMAX®; see Corrosion Information, page 14-15.

INSTALLATION: • Use all specified fasteners. See General Notes.

- See our *Anchoring and Fastening Systems for Concrete and Masonry* catalog, or visit ***www.strongtie.com*** for retrofit anchor options or reference technical bulletin T-ANCHORSPEC.
- Post bases do not provide adequate resistance to prevent members from rotating about the base and therefore are not recommended for non top-supported installations *(such as fences or unbraced carports)*.
- Place the base, load transfer plate and nut on the anchor bolt. Loosely tighten the nut.

ABW—Place the standoff base and then the post in the ABW and fasten on three vertical sides, using nails or Strong-Drive SD structural-connector screws.

- Make any necessary adjustments to post placement and tighten the nut securely on the anchor bolt.
- Bend up the fourth side of the ABW and fasten using the correct fasteners.

ABU—Place the standoff base and then the post in the ABU.

- Fasten using nails or Strong-Drive SD structural connector screws or bolts *(ABU88, ABU1010 – SDS optional)*.

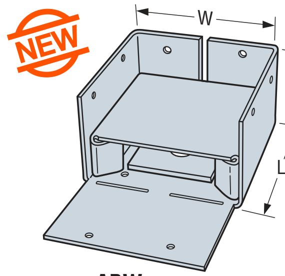
ABA—Place the post in the ABA.

- Fasten using nails or SD Screws.

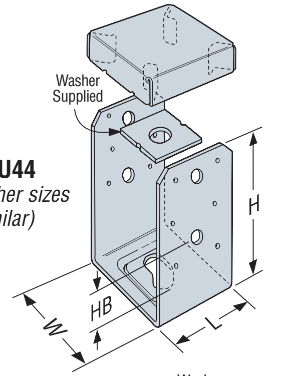
CODES: See page 13 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

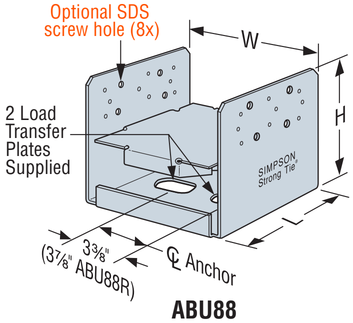
These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 27 for more information.



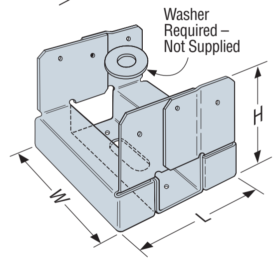
ABW



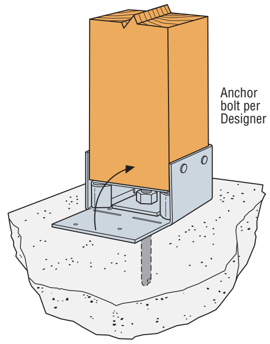
ABU44 *(other sizes similar)*



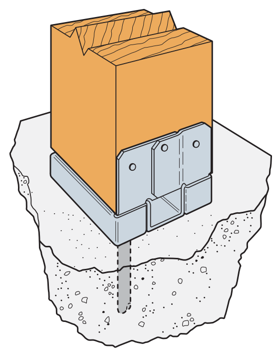
ABU88 *(ABU1010 similar)*



ABA44 *(other sizes similar)*



Typical ABW Installation



Typical ABA44 Installation

Model No.	Nominal Post Size	Material: Base (Ga)	Material: Strap (Ga)	Dimensions (in.): W	L	H	HB[6]	Fasteners: Anchor Dia. (in.)	Post: Nails	Machine Bolts: Qty.	Machine Bolts: Dia.	Allowable Loads (DF/SP): Uplift (160): Nails	Uplift (160): Bolts	Down (100)	Code Ref.
ABA44Z	4x4	16	16	3 9/16	3⅛	3 1/16	—	½	6-10d	—	—	555	—	6000	I3, F1
ABW44Z	4x4	16	16	3 9/16	3 9/16	2¼	—	½	8-10d	—	—	1005	—	7180	170
ABU44	4x4	16	12	3 9/16	3	5½	1¾	⅝	12-16d	2	½	2200	2160	6665	I3, L2, F1
ABA44R	Rough 4x4	16	16	4 1/16	3⅛	2 13/16	—	½	6-10d	—	—	555	—	8000	I3, F1
ABW44RZ	Rough 4x4	16	16	4	4 1/16	1 32/33	—	½	8-10d	—	—	1005	—	7180	170
ABW46Z	4x6	12	16	3 9/16	5 9/16	3	—	½	10-10d	—	—	845	—	4590	170
ABA46Z	4x6	14	14	3 9/16	5 3/16	3⅛	—	⅝	8-16d	—	—	700	—	9435	I3, F1
ABU46	4x6	12	12	3 9/16	5	7	2⅝	⅝	12-16d	2	½	2300	2300	10335	I3, L2, F1
ABW46RZ	Rough 4x6	12	16	4	6	2 13/16	—	½	10-10d	—	—	845	—	4590	170
ABA46R	Rough 4x6	14	14	4 1/16	5 3/16	2⅞	—	⅝	8-16d	—	—	700	—	12000	I3, F1
ABU5-5	5⅛x5⅛	12	10	5¼	5	6 1/16	1¾	⅝	12-16d	2	½	2235	2235	12000	
ABU5-6	5⅛x6	12	10	6⅛	5	6 1/16	1¾	⅝	12-16d	2	½	2235	2235	12000	
ABA66Z	6x6	14	14	5½	5¼	3⅛	—	⅝	8-16d	—	—	720	—	10665	I3, F1
ABW66Z	6x6	12	14	5½	5 9/16	3	—	½	12-10d	—	—	1190	—	12935	170
ABU66	6x6	12	10	5½	5	6 1/16	1¾	⅝	12-16d	2	½	2300	2300	12000	I3, L2, F1
ABA66R	Rough 6x6	14	14	6	5 3/16	2⅞	—	⅝	8-16d	—	—	720	—	12665	I3, F1
ABW66RZ	Rough 6x6	12	14	6	6	2 13/16	—	½	12-10d	—	—	1190	—	12935	170
ABU88[4]	8x8	14	12	7½	7	7	—	2-⅝	18-16d	—	—	2320	—	24335	I3, F1
ABU88R[4]	Rough 8x8	14	12	8	7	7	—	2-⅝	18-16d	—	—	2320	—	24335	170
ABU1010Z	10x10	12	12	9½	9	7¼	—	2-⅝	22-16d	—	—	2270	—	32020	
ABU1010RZ	Rough 10x10	12	12	10	9	7	—	2-⅝	22-16d	—	—	2270	—	32020	

1. Uplift loads have been increased for wind or earthquake with no further increase allowed; reduce where other loads govern.
2. Downloads may not be increased for short-term loading.
3. Specifier to design concrete for uplift capacity.
4. ABU products may be installed with either bolts or nails *(not both)* to achieve table loads. ABU88 and ABU88R may be installed with 8-SDS ¼"x3" wood screws *(sold separately)* for the same table load.
5. For AB bases, higher download can be achieved by solidly packing grout under 1" standoff plate before installation. Base download on column, grout, or concrete according to the code.
6. HB dimension is the distance from the bottom of the post up to the first bolt hole.
7. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers. For SCL columns, the fasteners for these products should always be installed in the wide face.
8. Downloads shall be reduced where limited by the capacity of the post. See pages 226-227 for common post allowable loads.
9. **NAILS:** 16d = 0.162" dia. x 3½" long, 10d = 0.148" dia. x 3" long. See page 22-23 for other nail sizes and information.

TOP FLANGE HANGERS – SOLID SAWN LUMBER (DF/SP)

SIMPSON Strong-Tie®

Joist or Purlin Size	Model No.	Ga	Dimensions W	Dimensions H	Dimensions B	Dimensions TF	Fasteners Header	Fasteners Joist	DF/SP Allowable Loads Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Installed Cost Index (ICI)	Code Ref.
SAWN LUMBER SIZES														
TPL 2x16	HU216-3TF	12	4 11/16	15	2½	2½	20-16d	8-16d	1765	5050	5050	5050	Lowest	I10, F9, L11
3x4	HU34TF	12	2 9/16	3 7/16	2½	2½	8-16d	2-10dx1½	295	2600	2600	2600	*	
3x6	W36	12	2 9/16	5⅜	2	2½	2-10d	2-10dx1½	—	2200	2200	2200	*	
	HU36TF	12	2 9/16	5⅜	2½	2½	10-16d	4-10dx1½	590	3725	3900	3900	*	
3x8	W38	12	2 9/16	7⅛	2	2½	2-10d	2-10dx1½	—	2200	2200	2200	*	
	HU38TF	12	2 9/16	7⅛	2½	2½	12-16d	4-10dx1½	590	3900	3900	3900	*	
	B38	12	2 9/16	7⅛	2½	2½	14-16d	6-16dx2½	1010	3800	3800	3800	*	I19, L13
3x10	W310	12	2 9/16	9⅛	2	2½	2-10d	2-10dx1½	—	2200	2200	2200	*	I10, F9, L11
	HU310TF	12	2 9/16	9⅛	2½	2½	14-16d	6-10dx1½	885	4170	4170	4170	*	
	B310	12	2 9/16	9⅛	2½	2½	14-16d	6-16dx2½	1010	3800	3800	3800	*	I19, F21, L13
3x12	WNP312	12	2 9/16	11	2½	2 3/16	2-10d	2-10dx1½	—	3255	3255	3255	*	I10, F9, L11
	HU312TF	12	2 9/16	11	2½	2½	16-16d	6-10dx1½	885	4335	4335	4335	*	
	B312	12	2 9/16	11	2½	2½	14-16d	6-16dx2½	1010	3800	3800	3800	*	I19, F21, L13
3x14	WNP314	12	2 9/16	13	2½	2 3/16	2-10d	2-10dx1½	—	3255	3255	3255	*	I10, F9, L11
	HU314TF	12	2 9/16	13	2½	2½	18-16d	8-10dx1½	1180	4335	4335	4335	*	
	B314	12	2 9/16	13	2½	2½	14-16d	6-16dx2½	1010	3800	3800	3800	*	I19, F21, L13
3x16	WNP316	12	2 9/16	15	2½	2 3/16	2-10d	2-10dx1½	—	3255	3255	3255	*	I10, F9, L11
	HU316TF	12	2 9/16	15	2½	2½	20-16d	8-10dx1½	1180	4335	4335	4335	*	
	B316	12	2 9/16	15	2½	2½	14-16d	6-16dx2½	1010	3800	3800	3800	*	I19, F21, L13
4x3	HU43TF	12	3 9/16	3	2½	2½	8-16d	2-10d	330	2600	2600	2600	*	I10, F9, L11
4x4	HU44TF	12	3 9/16	3 7/16	2½	2½	8-16d	2-10d	375	2600	2600	2600	Lowest	
4x6	HUS46TF	14	3 9/16	5⅜	2	1½	6-16d	4-16d	1235	2700	2890	3000	Lowest	
	W46	12	3 9/16	5⅜	2½	2½	2-10d	2-10d	—	2200	2200	2200	+12%	
	HU46TF	12	3 9/16	5⅜	2½	2½	10-16d	4-10d	750	3165	3165	3165	+28%	
	HW46	11	3 9/16	5⅜	2½	2½	4-10d	2-10d	—	5285	5285	5285	+83%	
4x8	BA48 (Min)	14	3 9/16	7⅛	3	2½	16-16d	2-10dx1½	265	3435	3435	3435	Lowest	I19, F21, L13
	BA48 (Max)	14	3 9/16	7⅛	3	2½	16-16d	8-10dx1½	1170	3800	3800	3800	+7%	
	HUS48TF	14	3 9/16	7¼	2	1 11/16	8-16d	6-16d	1550	3225	3495	3670	+33%	I10, IL12, F9, L11
	B48	12	3 9/16	7⅛	2½	2½	14-16d	6-16d	1010	3800	3800	3800	+35%	I19, F21, L13
	W48	12	3 9/16	7⅛	2½	2½	2-10d	2-10d	—	2200	2200	2200	+54%	I10, F9, L11
	HU48TF	12	3 9/16	7⅛	2½	2½	12-16d	4-10d	750	3500	3500	3500	+95%	
	HW48	11	3 9/16	7⅛	2½	2½	4-10d	2-10d	—	5285	5285	5285	+130%	
4x10	BA410 (Min)	14	3 9/16	9⅛	3	2½	16-16d	2-10dx1½	265	3435	3435	3435	Lowest	I19, F21
	BA410 (Max)	14	3 9/16	9⅛	3	2½	16-16d	8-10dx1½	1170	3800	3800	3800	+7%	
	HUS410TF	14	3 9/16	9¼	2	1½	10-16d	8-16d	2590	3365	3710	3935	+21%	I10, F9, L11
	B410	12	3 9/16	9⅛	2½	2½	14-16d	6-16d	1010	3800	3800	3800	+35%	I19, F21, L13
	W410	12	3 9/16	9⅛	2½	2½	2-10d	2-10d	—	2200	2200	2200	+49%	I10, F9, L11
	HU410TF	12	3 9/16	9⅛	2½	2½	14-16d	6-10d	1125	4150	4150	4150	+86%	
	HW410	11	3 9/16	9⅛	2½	2½	4-10d	2-10d	—	5285	5285	5285	+130%	
	GLT4	7	3 9/16	7½ Min.	5	2½	10-N54A	6-N54A	1865	7000	7000	7000	*	I19, F18, L14
	HGLT4	7	3 9/16	7½ Min.	6	2½	18-N54A	6-N54A	1865	12750	12750	12750	*	I19, L14
4x12	BA412 (Min)	14	3 9/16	11	3	2½	16-16d	2-10dx1½	265	3435	3435	3435	Lowest	I19, F21, L13
	BA412 (Max)	14	3 9/16	11	3	2½	16-16d	8-10dx1½	1170	3800	3800	3800	+6%	
	HUS412TF	14	3 9/16	11⅛	2	2	10-16d	8-16d	2000	4420	4760	4990	+14%	I10, F9
	B412	12	3 9/16	11	2½	2½	14-16d	6-16d	1010	3800	3800	3800	+27%	I19, F21, L13
	WNP412	12	3 9/16	11	2½	2 3/16	2-10d	2-10d	—	3255	3255	3255	+32%	I10, F9, L11
	HU412TF	12	3 9/16	11	2½	2½	16-16d	6-10d	1125	4550	4885	5105	+84%	
	HW412	11	3 9/16	11	2½	2½	4-10d	2-10d	—	5285	5285	5285	+115%	
	HHB412	7	3 9/16	11	3	2½	4-N54A	2-N54A	650	4185	4185	4185	+174%	I19, F18, L14
	GLT4	7	3 9/16	7½ Min.	5	2½	10-N54A	6-N54A	1865	7000	7000	7000	*	
	HGLT4	7	3 9/16	7½ Min.	6	2½	18-N54A	6-N54A	1865	12750	12750	12750	*	I19, L14

See footnotes on page 80.

CODES: See page 13 for Code Reference Key Chart.

LSU/LSSU *Adjustable Light Slopeable/Skewable U Hangers*

SIMPSON Strong-Tie®

This product is preferable to similar connectors because of a) easier installation, b) higher loads, c) lower installed cost, or a combination of these features.

The LSU and LSSU series of hangers may be sloped and skewed in the field, offering a versatile solution for attaching joists and rafters. These hangers may be sloped up or down and skewed left or right, up to 45°.

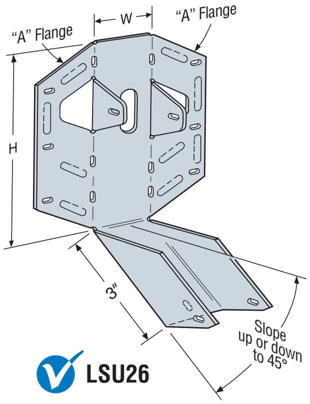
MATERIAL: See table

FINISH: Galvanized. Some products available in ZMAX® coating; see Corrosion Information, page 14-15.

INSTALLATION:

- Use all specified fasteners. See General Notes.
- Attach the sloped joist at both ends so that the horizontal force developed by the slope is fully supported by the supporting members.
- To see an installation video on this product, visit ***www.strongtie.com***.

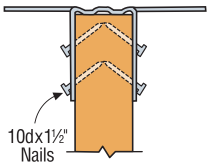
CODES: See page 13 for Code Reference Key Chart.



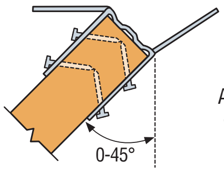
LSU26

LSU and LSSU INSTALLATION SEQUENCE

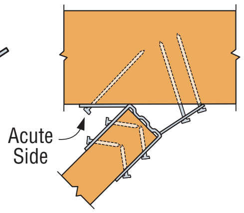
(For Skewed or Sloped/Skewed Applications)



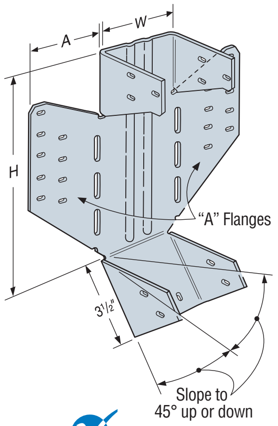
STEP 1
Nail hanger to slope-cut carried member, installing seat nail first. No bevel necessary for skewed installation. Install joist nails at 45° angle.



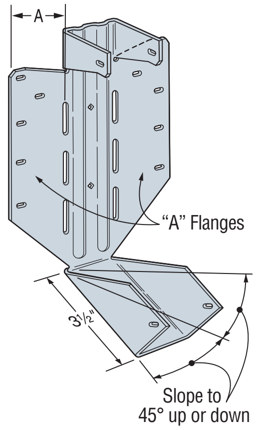
STEP 2
Skew flange from 0-45°. Bend other flange back along centerline of slots until it meets the header. Bend one time only.



STEP 3
Attach hanger to the carrying member, acute angle side first *(see footnote 4)*. Install nails at an angle.



LSSU410
(LSSU210-2 similar)

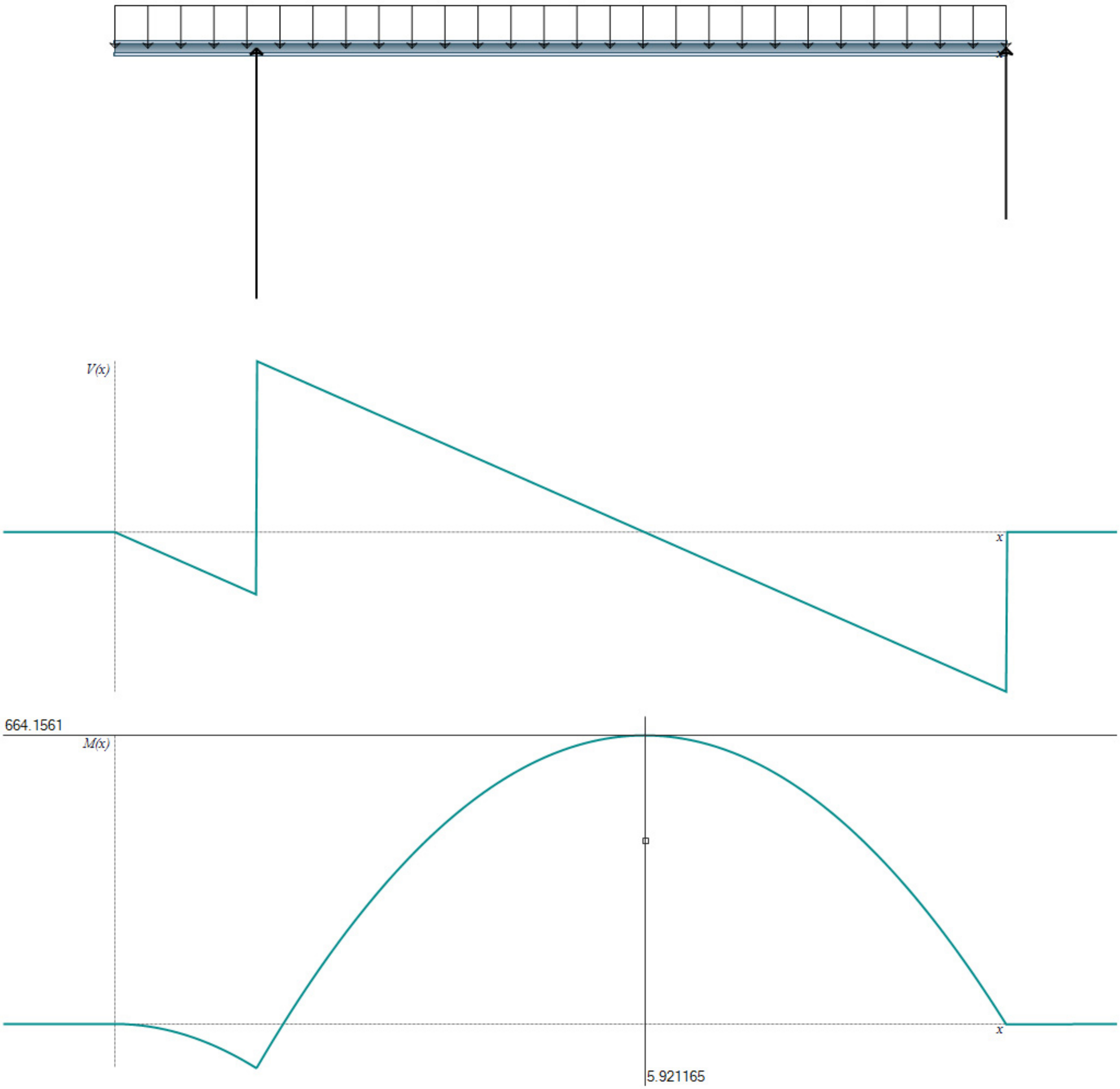


LSSU28

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Joist Width	Model No.	Ga	Dimensions			Fasteners		DF/SP Allowable Loads				SPF/HF Allowable Loads				Code Ref.
			W	H	A	Face	Joist	Uplift[2] (160)	Floor (100)	Snow (115)	Roof (125)	Uplift[2] (160)	Floor (100)	Snow (115)	Roof (125)	
Sloped Only Hangers																
1½	LSU26	18	1 9/16	4⅞	1½	6-10d	5-10dx1½	535	695	810	865	415	600	695	745	I8, F7, L15
1½	LSSU28	18	1 9/16	7⅛	1½	10-10d	5-10dx1½	535	1110	1275	1390	415	960	1105	1200	
1½	LSSU210	18	1 9/16	8½	1⅝	10-10d	7-10dx1½	875	1110	1275	1390	625	960	1105	1200	
2½	LSSUH310	16	2 9/16	8½	3⅛	18-16d	12-10dx1½	1150	2295	2295	2295	990	1930	1930	1930	170
3	LSSU210-2	16	3⅛	8½	2⅞	18-16d	12-10dx1½	1150	2430	2795	3035	990	2160	2485	2700	I8, F7, L15
3½	LSSU410	16	3 9/16	8½	2⅝	18-16d	12-10dx1½	1150	2430	2795	3035	990	2160	2485	2700	
Skewed Hangers or Sloped and Skewed																
1½	LSU26	18	1 9/16	4⅞	1½	6-10d	5-10dx1½	535	695	810	865	415	600	695	745	I8, F7, L15
1½	LSSU28	18	1 9/16	7⅛	1½	9-10d	5-10dx1½	450	885	885	885	415	765	765	765	
1½	LSSU210	18	1 9/16	8½	1⅝	9-10d	7-10dx1½	785	995	1145	1205	625	860	995	1050	
2½	LSSUH310	16	2 9/16	8½	3⅛	14-16d	12-10dx1½	1150	1600	1600	1600	990	1385	1385	1385	170
3	LSSU210-2	16	3⅛	8½	2⅞	14-16d	12-10dx1½	1150	1625	1625	1625	990	1365	1365	1365	I8, F7, L15
3½	LSSU410	16	3 9/16	8½	2⅝	14-16d	12-10dx1½	1150	1625	1625	1625	990	1365	1365	1365	

1. Roof loads are 125% of floor loads unless limited by other criteria.
2. Uplift loads include an increase for wind or earthquake loading with no further increase allowed; reduce when other loads govern.
3. Truss chord cross-grain tension may limit allowable loads in accordance with ANSI/TPI 1-2007. Simpson Strong-Tie® Connector Selector™ Software includes the evaluation of cross-grain tension in its hanger allowable loads. For additional information, contact Simpson Strong-Tie.
4. For skewed LSSU hangers, the inner most face fasteners on the acute angle side are not installed.
5. Do not substitute 10dx1½" nails for face nails on slope and skew combinations or skewed only LSU and LSSU.
6. **NAILS:** 16d = 0.162" dia. x 3½" long, 10d = 0.148" dia. x 3" long, 10dx1½ = 0.148" dia. x 1½" long. See page 22-23 for other nail sizes and information.



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f 329.8 9.95

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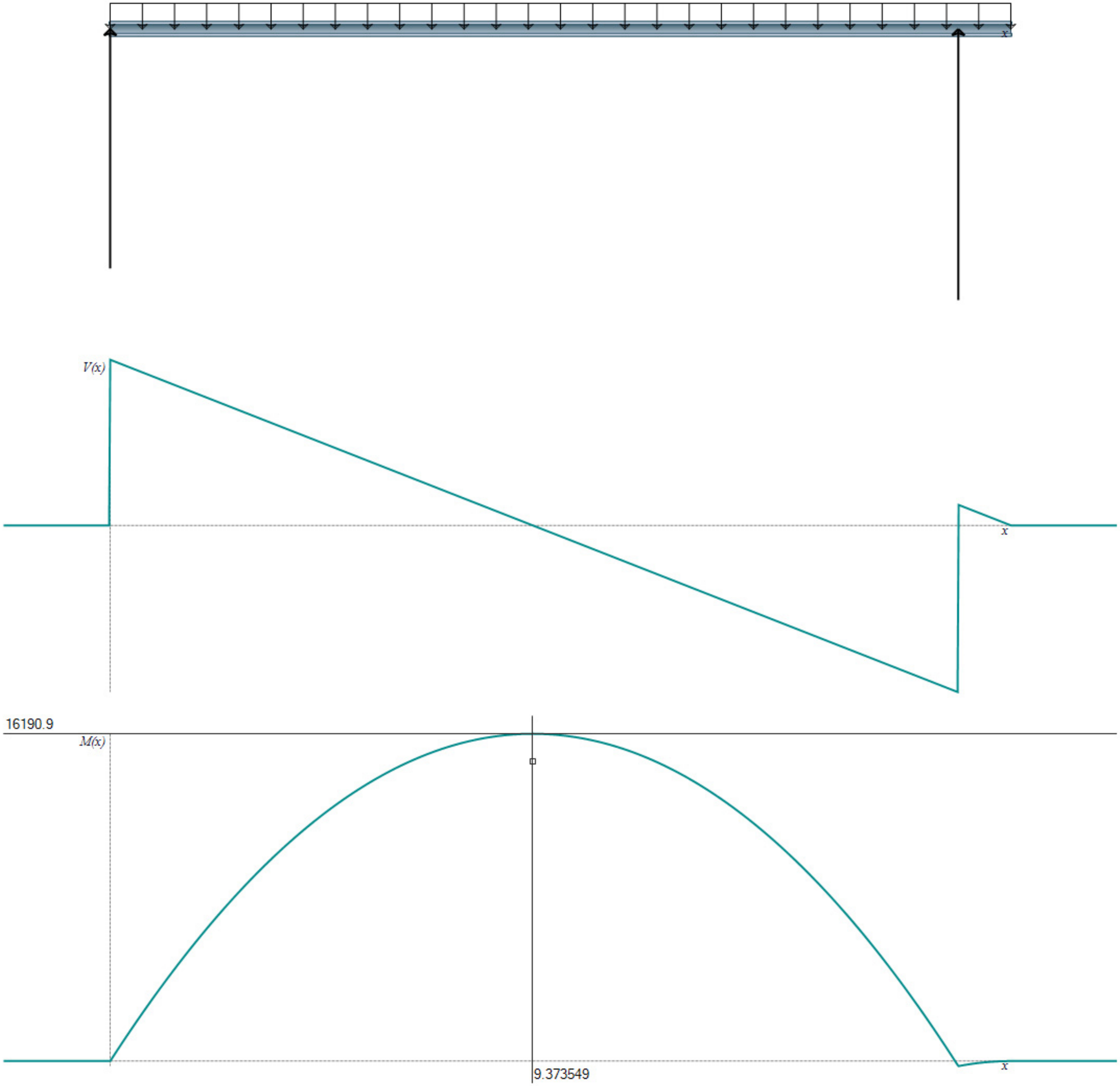

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f 3907.97 18.833

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