4227 Mercerwood Addition

Project Number: 24-065

4227 91st Ave SE

Mercer Island, WA 98040

Structural Calculations

8/1/24

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August 1st, 2024

PROJECT: Mercerwood Addition

DESIGNER: NKH & AKR

DATE: July 31st, 2024

JOB #: 24 - 065

PROJECT SUMMARY & DESIGN CRITERIA

Background
Project Summary:
This is a two story addition on an existing two story, wood framed house for Hui Zhou in Seattle, WA. The structure consists of wood roof & floor trusses/joists bearing on wood framed walls, posts, & beams. The house is supported by existing & new concrete stem walls & shallow spread footings. This project is designed in accordance with the 2021 International Building Code along with the codes listed below and corresponding state & city/county amendments.

Notes:
All input variables are highlighted in yellow, resources bolded, and links to resources bolded and underlined. Areas highlighted in blue are code/design checks and green - unity checks.

Resources:
- American Wood Council (AWC). (2018). "National Design Specifications for Wood Construction (NDS)."
- American Wood Council (AWC). (2021). "Special Design Provisions for Wind and Seismic (SDWS)."
- American Concrete Institute (ACI). (2019). "Building Code Requirements for Structural Concrete (ACI 318-19)."
- American Institute of Steel Construction (AISC). (2016). "Steel Construction Manual." 15th Ed.
- American Society of Civil Engineers (ASCE). (2022). "Minimum Design Loads for Buildings and Other Structures."
- State of Washington (2021). "International Building Code (IBC)."
- American Society of Civil Engineering (ASCE). "ASCE Hazard Tool"https://asce7hazardtool.online/

Material Properties

Soil:
-Soil Bearing Pressure (min per IBC1806.2) $p_{brg} := 1500\ psf$
-Frost Depth $FD := 18\ in$
- Active & Passive Soil Pressure $q_a := 35 \cdot pcf$ $q_p := 250 \cdot pcf$

Concrete:
-Compressive Strength $f'_c := 2500\ psi$
-Density, Normal Weight $\gamma_{conc} := 150 \cdot pcf$
-Density, Light Weight $\gamma_{conc\ LW} := 115 \cdot pcf$
-Reinforcing Steel, ASTM A615 $f_{yr} := 60\ ksi$

Steel:
- Modulus of Elasticity $E_c := 29000\ ksi$
-Anchor Rods/Bolts, ASTM A307 Shear & Tension Yield Strength $F_{nv} := 24\ ks$ $F_{nt} := 45\ ksi$

Wood:
-Solid Sawn Joists, Beams, Headers, & Studs DF-L #1 6x & Larger, DF-L #2 All Other (UNO)
-Glulam Beams 24F-V4 (Simple Span), 24F-V8 (Cont/Cantilever)

Gravity Loading

Roof Dead Load

Roofing	$R := 1.5\ psf$	
Insulation	$I := 2.0\ psf$	
Ceiling	$C := 2\ psf$	
Sheathing	$t := 0.5\ in$	$SH := t \cdot \left(\frac{3.25\ psf}{1\ in} \right) = 1.625\ psf$
Structural Members	$S := 2.5\ psf$	
Lights	$L := 1\ psf$	
Mechanical	$M := 1.5\ psf$	
Misc.	$MISC := 2.9\ psf$	

$DL_{rf} := R + I + C + SH + S + L + M + MISC$ $DL_{rf} = 15\ psf$ $DL_{pv} := 0\ psf$

Seismic Roof Dead Load

$SDL_{rf} := DL_{rf} - MISC + DL_{pv} \cdot (75\ \%) = 12.1\ psf$ $SDL_{rf} = 12\ psf$

Floor Dead Load

Flooring	$F := 1.5\ psf$	
Insulation	$I := 2.0\ psf$	
Ceiling	$C := 0\ psf$	
Sheathing	$t := 0.75\ in$	$SH := t \cdot \left(\frac{3.25\ psf}{1\ in} \right) = 2.4375\ psf$
Structural Members	$S := 3.4\ psf$	
Lights	$L := 1\ psf$	
Mechanical	$M := 1.5\ psf$	
Misc.	$MISC := 3.2\ psf$	

$DL_{flr} := F + I + C + SH + S + L + M + MISC$ $DL_{flr} = 15\ psf$

Seismic Floor Dead Load

$SDL_{flr} := DL_{flr} = 719.9994\ Pa$ $SDL_{flr} = 15\ psf$

Wall Dead Loads

Exterior Wood	$p_{ext\_w} := 10\ psf$
Interior Wood	$p_{int} := 9\ psf$

Live Loads

Roof	$LL_{rf} := 20\ psf$	Roof Snow Load	$SL := 25\ psf$
Floor Live Load	$LL_{flr} := 40\ psf$		
Deck Live Load	$LL_{deck} := 1.5 \cdot LL_{flr} = 2872.8155\ Pa$		

Deflection Criteria

$\Delta_{rf\_TL} := \frac{L}{240}$ $\Delta_{rf\_LL} := \frac{L}{360}$ $\Delta_{flr\_TL} := \frac{L}{360}$ $\Delta_{flr\_LL} := \frac{L}{480}$

S2

NKH ENGINEERING

PROJECT: Mercerwood Addition

DESIGNER: NKH & AKR

DATE: August 1st, 2024

JOB #: 24-065

LATERAL ANALYSIS

General

Input *Output* *Verify*

LRFD

Risk Cat.: II (ref. 1.5-1)

$L := 59\ ft$	Building Length	$SDL_{rf} := 12\ psf$	Roof Seismic Dead Load
$B := 51\ ft$	Building Width	$SDL_{flr} := 15\ psf$	Floor Seismic Dead Load
$h_{rf} := 24\ ft$	Avg Roof Height Upper	$LL := 25\ psf$	Roof Snow/Live Load
$h_p := 0\ ft$	Parapet Height	$p_{ext} := 12\ psf$	Exterior Wall Load
$h_{wall} := 10\ ft$	Wall Height	$p_{int} := 10\ psf$	Interior Wall Load

$a := \min\left(\begin{bmatrix} 10\ \% \cdot B \\ 0.4 \cdot h_{rf} \end{bmatrix}\right) = 5.1\ ft$ Width of Pressure Coefficient Zone

MWFRS (per ASCE 7 - 22, Chapter 26 & 27)

$theta := \operatorname{atan}\left(\frac{10}{12}\right) = 39.81\ deg$ Roof Slope

Design Velocity Pressure - Enclosed/Partially Enlosed Buildings

$V_w := 110$ mph — Basic Wind Speed (ref. figure 26.5-1A & city/county design criteria)

$K_d := 0.85$ — Directionality Factor (ref. section 26.6 & table 26.6 - 1)

exp = "B" — Exposure Category (ref. section 26.7)

$K_{zt} := 1.38$ — Topographic Factor (ref. section 26.8)

$K_z := 0.66$ — Velocity Pressure Exposure Coefficient (ref. table 26.10-1)

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V_w^{\ 2} \cdot (psf)$ Velocity pressure (eq 27.3-1) $q_z = 24\ psf$

Design Wind Pressure

$p_{w\_min} := 16\ psf$ — Minimum Design Pressure

$G_e := 0.85$ — Gust Effect Factor (ref. section 26.9)

Walls

$GC_{pi} := 0.18$ — Internal Pressure Coefficient (ref. table 26.11-1)

Veolcity Pressure Evaluated at Mean Roof Height, h

$q_h := q_z = 23.98\ psf$

S3

Design Wind Pressure (cont'd)

External Pressure Coefficients for Walls (ref. figure 27.3-1)

$\frac{L}{B} = 1.1569$ $C_{pww} := 0.8$ Windward Wall $C_{plw} := -0.25$ Leeward Wall

$C_p := C_{pww} - C_{plw} = 1.05$

Design MWFRS Wind Pressures (eq 27.3-1)

$$p_w := \max\left(\begin{bmatrix} p_{w\_min} \\ q_h \cdot \left(G_e \cdot C_p + GC_{pi}\right) \\ q_h \cdot \left(G_e \cdot C_p - GC_{pi}\right) \end{bmatrix}\right)$$

$p_w = 25.72\ \text{psf}$

$q_h \cdot \left(G_e \cdot C_p - GC_{pi}\right) = 17.09\ \text{psf}$

Parapet (ref. section 27.4.5)

$GC_{pnw} := 1.5$ Windward Combined Net Pressure Coefficient

$GC_{pnL} := -1.0$ Leeward Combined Net Pressure Coefficient

$p_p := \text{if } h_p \le 0$
$\quad 0\ \text{psf}$
else
$\quad q_z \cdot \left(GC_{pnw} - GC_{pnL}\right)$

Combined Net Pressure on Parapet $p_p = 0\ \text{psf}$

Roof (fig. 27.4-1) $\frac{h_{rf}}{L} = 0.4068$

$GC_{pi} = 0.18$ Internal pressure coefficient (ref. table 26.11-1)

External pressure coefficients for roofs (ref. figure 27.4-1)

$\frac{h_{rf}}{L} = 0.4068$ $C_{prf1} := -0.6$ $C_{prf2} := 0.2$

Veolcity pressure evaluated at mean roof height, h

$q_h = 23.98\ \text{psf}$

Design MWFRS wind pressure (ref. eq 27.4-1)

$p_{rf\_pos1} := q_h \cdot \left(G_e \cdot C_{prf1} + GC_{pi}\right) = -7.91\ \text{psf}$ $p_{rf\_pos2} := q_h \cdot \left(G_e \cdot C_{prf2} + GC_{pi}\right) = 8.39\ \text{psf}$

$p_{rf\_neg1} := q_h \cdot \left(G_e \cdot C_{prf1} - GC_{pi}\right) = -16.55\ \text{psf}$ $p_{rf\_neg2} := q_h \cdot \left(G_e \cdot C_{prf2} - GC_{pi}\right) = -0.24\ \text{psf}$

$p_{rf} := \left|p_{rf\_neg1}\right| = 16.5469\ \text{psf}$ $p_{rf\_horiz} := p_{rf} \cdot \sin\left(theta\right) = 10.5931\ \text{psf}$

$$p_{w\_up} := 0.6 \cdot SDL_{rf} + 0.6 \cdot \left(\min\left(\begin{bmatrix} p_{rf\_neg1} \\ p_{rf\_neg2} \end{bmatrix}\right)\right)$$

Net uplift pressure (ASD) $p_{w\_up} = -2.73\ \text{psf}$

Roof Overhangs

$C_{poh} := -0.8$ External pressure coefficients for roof overhangs (ref. 27.5.3)

$p_{oh} := q_z \cdot \left(G_e \cdot C_{poh}\right) + \min\left(\begin{bmatrix} p_{rf\_neg1} \\ p_{rf\_neg2} \end{bmatrix}\right)$ Overhang pressure $p_{oh} = -32.85\ \text{psf}$

$OH_{net} := 0.6 \cdot SDL_{rf} + 0.6 \cdot p_{oh}$ Net uplift pressure (ASD) $OH_{net} = -12.51\ \text{psf}$

S4

C&C (per ASCE 7 - 22, Chapter 30)

Walls (ref. eq. 30.4-1 & figure 30.4-1)

$GC_{pw4\_pos} := 1.0$ $GC_{pw4\_neg} := -1.1$ exterior pressure coefficients

$GC_{pw5\_pos} := 1.0$ $GC_{pw5\_neg} := -1.4$ exterior pressure coefficients (corner zone)

$p_{cc\_w4pos} := q_h \cdot \left(GC_{pw4\_pos} + GC_{pi} \right) = 28.3\ psf$ Positive design wind pressure (ref. eq. 30.4-1)

$p_{cc\_w4neg} := q_h \cdot \left(GC_{pw4\_neg} - GC_{pi} \right) = -30.7\ psf$ Negative design wind pressure

$p_{cc\_w5pos} := q_h \cdot \left(GC_{pw5\_pos} + GC_{pi} \right) = 28.3\ psf$ Corner zone positive design wind pressure

$p_{cc\_w5neg} := q_h \cdot \left(GC_{pw5\_neg} - GC_{pi} \right) = -37.89\ psf$ Corner zone negative design wind pressure

Roofs (ref. eq. 30.4-1 & figure 30.4-2B)

Negative design wind pressure $GC_{pr1} := -0.9$ $GC_{pr2} := -1.7$ $GC_{pr3} := -2.6$

$p_{cc\_r1} := q_h \cdot \left(GC_{pr1} - GC_{pi} \right) = -25.9\ psf$ $p_{cc\_r3} := q_h \cdot \left(GC_{pr3} - GC_{pi} \right) = -66.67\ psf$

$p_{cc\_r2} := q_h \cdot \left(GC_{pr2} - GC_{pi} \right) = -45.08\ psf$

Positive design wind pressure $GC_{pr\_pos} := 0.5$

$p_{cc\_r2} := q_h \cdot \left(GC_{pr\_pos} + GC_{pi} \right) = 16.31\ psf$

Wind Diaphragm Shear (LRFD)

$A_{wall\_L} := 1134\ ft^2$ $A_{roof\_L} := 470\ ft^2$ $A_{wall\_T} := 776\ ft^2$ $A_{roof\_T} := 538\ ft^2$

$V_{wu\_L} := p_w \cdot A_{wall\_L} + p_{rf\_horiz} \cdot A_{roof\_L} = 34.14\ kip$ Longitudinal diaphragm shear

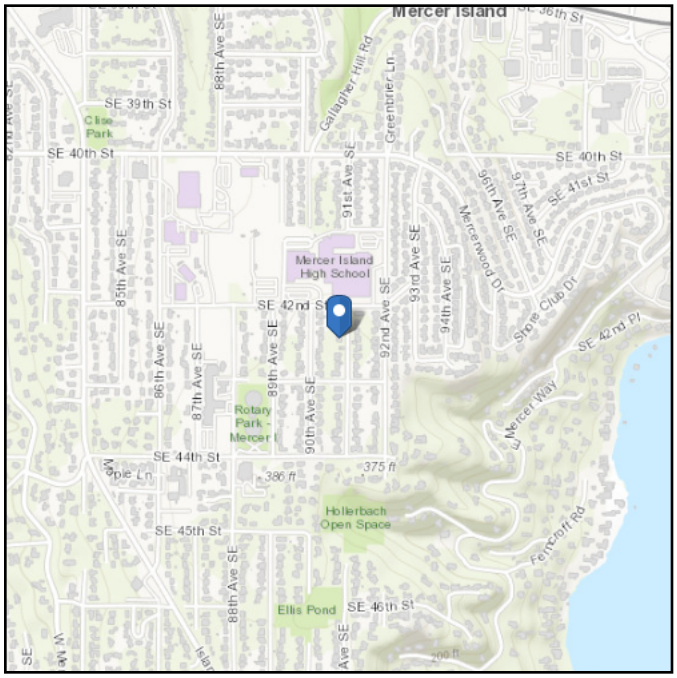
$V_{wu\_T} := p_w \cdot A_{wall\_T} + p_{rf\_horiz} \cdot A_{roof\_T} = 25.66\ kip$ Transverse diaphragm shear

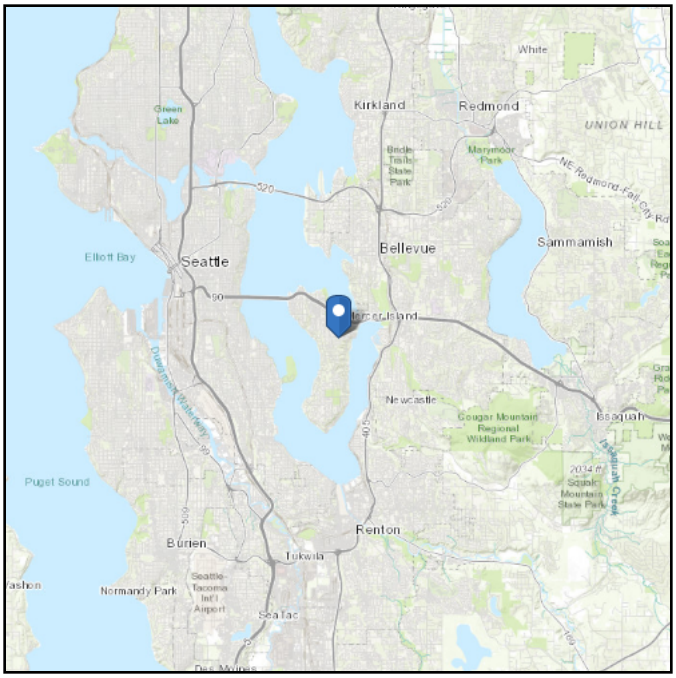
S5

ASCE Hazards Report

Address:
4227 91st Ave SE
Mercer Island, Washington
98040

Standard:	ASCE/SEI 7-22	**Latitude:**	47.570131
Risk Category:	II	**Longitude:**	-122.217832
Soil Class:	Default	**Elevation:**	369.4426286823999 ft (NAVD 88)





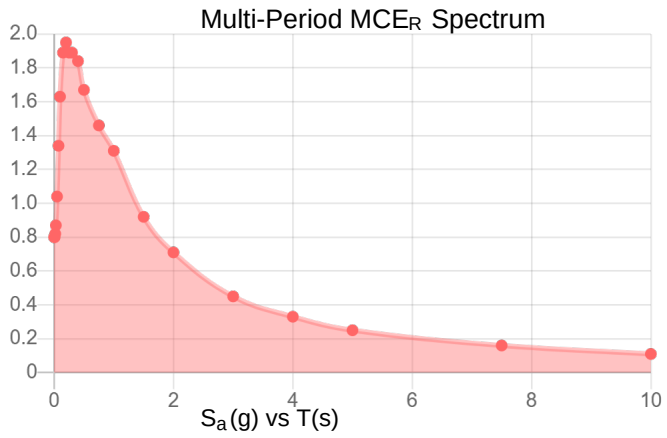
Seismic

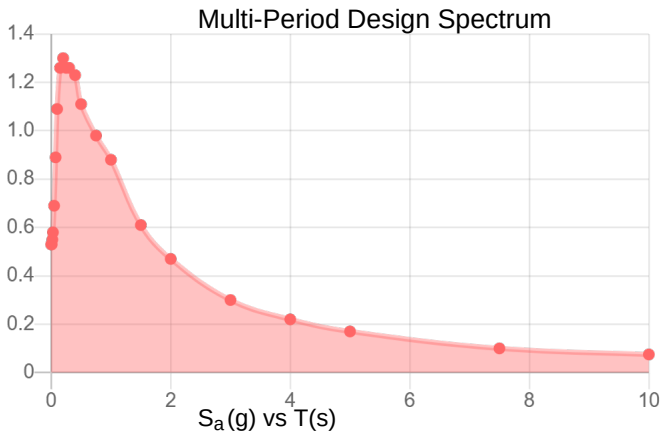
Site Soil Class: Default

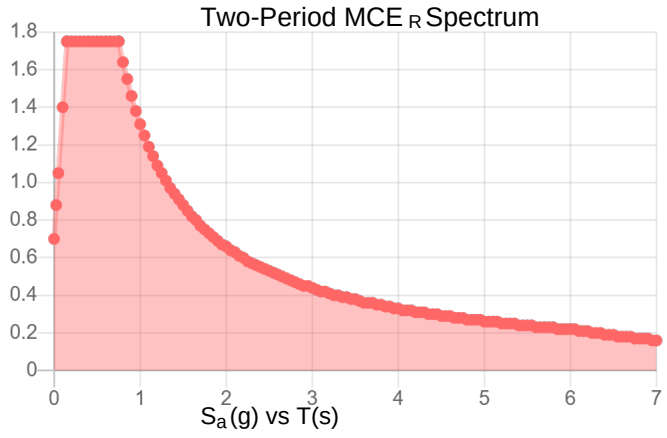
Results:

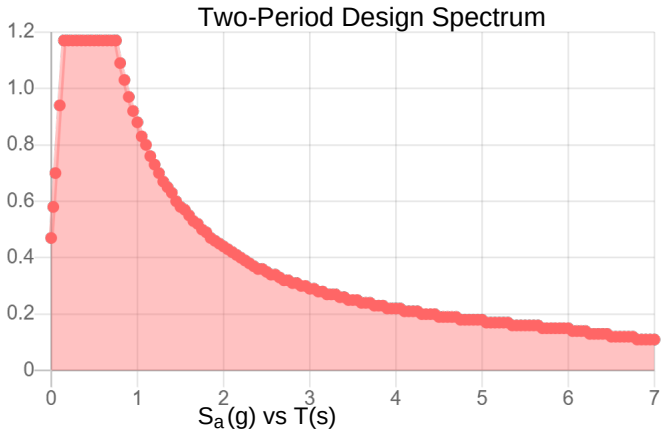
PGA_M :	0.73	T_L :	6
S_{MS} :	1.75	S_S :	1.57
S_{M1} :	1.31	S_1 :	0.63
S_{DS} :	1.17	V_{S30} :	260
S_{D1} :	0.88		

Seismic Design Category: D









MCER Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Seismic (per ASCE 7 - 22, 12.8)

LRFD

Basic Parameters

- Equivalent Lateral Force Procedure (ELFP)
- Site class: D
- Seismic design category: D
- Light Framed Wood Walls Sheathed w/ Wood Panels

$I_s := 1.0$ Seismic importance factor (ref. table 1.5-2)

$S_{DS} := 1.17$ Design spectral acceleration parameter (ref. ASCE Summary Report)

$R := 6.5$ Response modification factor - (ref. table 12.2-1)

$\Omega_o := 3$ System overstrength factor (ref. table 12.2-1)

$C_d := 4$ Deflection amp. factor (ref. table 12.2-1)

$\rho := 1.0$ Redundancy factor (ref section 12.3.4)

$C_s := \frac{S_{DS}}{\left(\frac{R}{I_s}\right)} = 0.18$ Seismic resonse coefficient (EQ. 12.8 - 2)

$S_{D1} := 0.88$ $S_1 := 0.63$

$h_n := h_{rf} = 24\ ft$ Highest level of structure $h_{in} := 10\ ft$ Interior wall height

$C_t := 0.02$ $x := 0.75$ Table 12.8-2

$T_a := C_t \cdot \left(\frac{h_n}{ft}\right)^x = 0.2169$ EQ 12.8-7

$C_{smax} := \frac{S_{D1}}{T_a \cdot \left(\frac{R}{I_s}\right)}$ $C_{smax} = 0.6243$

$C_s := \max\left(\left[\begin{matrix} 0.1 \\ \min\left(\left[\begin{matrix} C_s \\ C_{smax} \end{matrix}\right]\right) \end{matrix}\right]\right)$ $C_s = 0.18$

$C_{s\_min} := \frac{0.5 \cdot S_{D1}}{\left(\frac{R}{I_s}\right)} = 0.0677$ EQ 12.8- 5 & 12.8- 6

$C_s := \rho \cdot C_s$ $C_s = 0.18$

S8

Seismic Diaphragm Shear

Building Weights Contributing to Seismic Forces

$W_{diaphragm\_rf} := 1707\ ft^2 \cdot SDL_{rf}$

$W_{diaphragm\_flr} := 2315\ ft^2 \cdot SDL_{flr}$

$W_{diaphragm} := W_{diaphragm\_rf} + W_{diaphragm\_flr}$

$W_{walls\_T} := \left(p_{ext} + p_{int}\right) \cdot A_{wall\_T} \cdot 2$

$W_{walls\_T} = 34.144\ kip$

$W_{walls\_L} := \left(p_{ext} + p_{int}\right) \cdot A_{wall\_L} \cdot 2$

$W_{walls\_L} = 49.896\ kip$

Shear Loads

$V_{su} := C_s \cdot \left(W_{diaphragm} + W_{walls\_L} + W_{walls\_T}\right)$

$V_{su} = 25.0648\ kip$

Lateral Summary

Seismic /Wind Shearwall Capacity Factor
(ref. NDS Shearwall Capacities)

$C_{sw\_cap} := \frac{310\ psf}{435\ psf} = 0.71$

	Wind	Seismic
Transverse	$V_{wu\_T} := 0.6 \cdot V_{wu\_T} \cdot C_{sw\_cap} = 10.97\ kip$	$V_{su\_L} := 0.7 \cdot V_{su} \cdot C_{sw\_cap} = 12.5\ kip$

```
V_T := if  V_wu_T > V_su_L        = "SEISMIC CONTROLS"
       "WIND CONTROLS"
     else
       "SEISMIC CONTROLS"
```

Longitudinal	$V_{wu\_L} := 0.6 \cdot V_{wu\_L} \cdot C_{sw\_cap} = 14.6\ kip$	$V_{su\_T} := 0.7 \cdot V_{su} \cdot C_{sw\_cap} = 12.5\ kip$

```
V_L := if  V_wu_L > V_su_T        = "WIND CONTROLS"
       "WIND CONTROLS"
     else
       "SEISMIC CONTROLS"
```

Wall Line Wind Force Reactions - Roof

$h_{wall} = 10\ ft$ Average Wall Height

$h_{rf\_proj} := 6\ ft$ Roof Projection above wall

$p_w = 25.72\ psf$ Design Wind Pressure (ref. 'Wind Loading')

$p_{rf\_horiz} = 10.59\ psf$ Design Wind Pressure (ref. 'Wind Loading')

Longitudinal Wall Line Reactions

Reaction B $tribB := \frac{29\ ft}{2} = 14.5\ ft$

$R_{Lrf\_B} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot tribB$ $R_{Lrf\_B} = 2.79\ kip$

Reaction F $tribF := \frac{28\ ft}{2} = 14\ ft$

$R_{Lrf\_F} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot (tribB + tribF)$ $R_{Lrf\_F} = 5.48\ kip$

Reaction H $tribH := tribF = 14\ ft$

$R_{Lrf\_H} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot (tribH)$ $R_{Lrf\_H} = 2.69\ kip$

Transverse Wall Line Reactions

Reaction 3 $trib3 := \frac{21\ ft}{2} + 4\ ft = 14.5\ ft$

$R_{Trf\_3} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot trib3$ $R_{Trf\_3} = 2.79\ kip$

Reaction 5 $trib5 := \frac{11\ ft}{2} = 5.5\ ft$

$R_{Trf\_5} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot ((trib3 - 4\ ft) + trib5)$ $R_{Trf\_5} = 3.07\ kip$

Reaction 6 $trib6 := trib5 = 5.5\ ft$

$R_{Trf\_6} := \left(p_w \cdot \frac{h_{wall}}{2} + p_{rf\_horiz} \cdot h_{rf\_proj}\right) \cdot (trib6)$ $R_{Trf\_6} = 1.06\ kip$

S10

Vertical Distribution of Seismic Forces - Roof Diaphragm (Per ASCE 7- 22, 12.8.3)

$V_{su} = 25.0648\ \text{kip}$

$$C_{vx\_roof} := \frac{\left(W_{diaphragm\_rf} + W_{walls\_L} \cdot \frac{1}{4} + W_{walls\_T} \cdot \frac{1}{4}\right)}{\left(W_{diaphragm} + W_{walls\_L} + W_{walls\_T}\right)} = 0.298$$

$V_{rf} := V_{su} \cdot C_{vx\_roof}$ Distributed shear to roof $V_{rf} = 7.47\ \text{kip}$

Wall Line Seismic Force Reactions - Roof

Longitudinal

Reaction B $R_{Lrf\_B\_EL} := \frac{tribB}{L} \cdot V_{rf}$ $R_{Lrf\_B\_EL} = 1.84\ \text{kip}$

Reaction F $R_{Lrf\_F\_EL} := \frac{tribF + tribB}{L} \cdot V_{rf}$ $R_{Lrf\_F\_EL} = 3.61\ \text{kip}$

Reaction H $R_{Lrf\_H\_EL} := \frac{tribH}{L} \cdot V_{rf}$ $R_{Lrf\_H\_EL} = 1.77\ \text{kip}$

Transverse

Reaction 3 $R_{Trf\_3\_EL} := \frac{trib3}{36\ \text{ft}} \cdot V_{rf}$ $R_{Trf\_3\_EL} = 3.01\ \text{kip}$

Reaction 5 $R_{Trf\_5\_EL} := \frac{trib3 + trib5}{36\ \text{ft}} \cdot V_{rf}$ $R_{Trf\_5\_EL} = 4.15\ \text{kip}$

Reaction 6 $R_{Trf\_6\_EL} := \frac{trib6}{36\ \text{ft}} \cdot V_{rf}$ $R_{Trf\_6\_EL} = 1.14\ \text{kip}$

Wall Line Wind Force Reactions - Upper Floor

$h_{wall} = 10\ ft$ Average Wall Height

$p_w = 25.72\ psf$ Design Wind Pressure (ref. 'Wind Loading')

Longitudinal Grid Line Reactions

Reaction B $tribB := \frac{29\ ft}{2} = 14.5\ ft$

$R_{Lup\_B} := (p_w \cdot h_{wall}) \cdot tribB + R_{Lrf\_B}$ $R_{Lup\_B} = 6.52\ kip$

Reaction F $tribF := \frac{14\ ft}{2} = 7\ ft$

$R_{Lup\_F} := (p_w \cdot h_{wall}) \cdot (tribB + tribF) + R_{Lrf\_F}$ $R_{Lup\_F} = 11.01\ kip$

Reaction G $tribG := \frac{14\ ft}{2} = 7\ ft$

$R_{Lup\_G} := (p_w \cdot h_{wall}) \cdot (tribF + tribG)$ $R_{Lup\_G} = 3.6\ kip$

Reaction H $tribH := tribG = 7\ ft$

$R_{Lup\_H} := (p_w \cdot h_{wall}) \cdot (tribH) + R_{Lrf\_H}$ $R_{Lup\_H} = 4.49\ kip$

Transverse Wall Line Reactions

Reaction 3 $trib3 := \frac{21\ ft}{2} + 6\ ft = 16.5\ ft$

$R_{Tup\_3} := (p_w \cdot h_{wall}) \cdot trib3 + R_{Trf\_3}$ $R_{Tup\_3} = 7.03\ kip$

Reaction 5 $trib5 := \frac{24\ ft}{2} = 12\ ft$

$R_{Tup\_5} := (p_w \cdot h_{wall}) \cdot ((trib3 - 6\ ft) + trib5) + R_{Trf\_5} + \frac{R_{Trf\_6}}{2}$ $R_{Tup\_5} = 9.39\ kip$

Reaction 7 $trib7 := trib5 = 12\ ft$

$R_{Tup\_7} := (p_w \cdot h_{wall}) \cdot (trib7) + \frac{R_{Trf\_6}}{2}$ $R_{Tup\_7} = 3.61\ kip$

S12

Vertical Distribution of Seismic Forces - Upper Floor Diaphragm (Per ASCE 7- 22, 12.8.3)

$V_{su} = 25.0648\ kip$

$$C_{vx\_up} := \frac{\left(W_{diaphragm\_flr} + W_{walls\_L}\cdot\frac{1}{2} + W_{walls\_T}\cdot\frac{1}{2}\right)}{\left(W_{diaphragm} + W_{walls\_L} + W_{walls\_T}\right)} = 0.5511$$

$V_{up} := V_{su}\cdot C_{vx\_roof}$ Distributed shear to roof $V_{up} = 7.47\ kip$

Wall Line Seismic Force Reactions - Upper Floor

Longitudinal

Reaction B $R_{Lup\_B\_EL} := \frac{tribB}{L}\cdot V_{up} + R_{Lrf\_B\_EL}$ $R_{Lup\_B\_EL} = 3.67\ kip$

Reaction F $R_{Lup\_F\_EL} := \frac{tribB + tribF}{L}\cdot V_{up} + R_{Lrf\_F\_EL}$ $R_{Lup\_F\_EL} = 6.33\ kip$

Reaction G $R_{Lup\_G\_EL} := \frac{tribF + tribG}{L}\cdot V_{up}$ $R_{Lup\_G\_EL} = 1.77\ kip$

Reaction H $R_{Lup\_H\_EL} := \frac{tribH}{L}\cdot V_{up} + R_{Lrf\_H\_EL}$ $R_{Lup\_H\_EL} = 2.66\ kip$

Transverse

Reaction 3 $R_{Tup\_3\_EL} := \frac{trib3}{B}\cdot V_{up} + R_{Trf\_3\_EL}$ $R_{Tup\_3\_EL} = 5.42\ kip$

Reaction 5 $R_{Tup\_5\_EL} := \frac{trib5}{B}\cdot V_{up} + R_{Trf\_5\_EL} + \frac{R_{Trf\_6\_EL}}{2}$ $R_{Tup\_5\_EL} = 6.48\ kip$

Reaction 7 $R_{Tup\_7\_EL} := \frac{trib7}{B}\cdot V_{up} + \frac{R_{Trf\_6\_EL}}{2}$ $R_{Tup\_7\_EL} = 2.33\ kip$

Diaphragm Check (ref. ANSI/AF&PA SDPWS-2021)

Aspect Ratio

$L_T := L = 59\ ft$ $L_L := B = 51\ ft$ Length & width of diaphragm

$check_D := \text{if } \frac{L_L}{L_T} > 4$ "NG" else "OK"

$ratio := \frac{L_L}{L_T} = 0.86$

$check_D = \text{"OK"}$

Diaphragm Shear

Shear capacities for 19/32" APA Rated OSB/Plywood Sheathing - Un-Blocked (ref. table 4.2A):

$\Omega_{D\_w} := 2.0$ Wind ASD reduction factor

$v_{w6} := 800 \cdot plf \cdot \frac{1}{\Omega_{D\_w}} = 400\ plf$ Allowable Shear Capacity- 10d's @ 6" oc

$\Omega_{D\_s} := 2.8$ Seismic ASD reduction factor

$v_{s6} := 800 \cdot plf \cdot \frac{1}{\Omega_{D\_s}} = 285.7\ plf$ Allowable Shear Capacity- 10d's @ 6" oc

Diaphragm

$V_{diaphT\_WL} := 0.6 \cdot R_{Tup\_5} = 5.63\ kip$ Wind diaphragm shear transverse direction

$V_{diaphL\_WL} := 0.6 \cdot R_{Lup\_F} = 6.6\ kip$ Wind diaphragm shear longitudinal direction

$V_{diaphT\_EL} := 0.7 \cdot R_{Tup\_5} = 6.57\ kip$ Seismic diaphragm shear transverse direction

$V_{diaphL\_EL} := 0.7 \cdot R_{Lup\_F} = 7.7\ kip$ Seismic diaphragm shear longitudinal direction

Longitudinal Diaphargm Shear

$v_{L\_w} := \frac{V_{diaphL\_WL}}{L_T} = 111.93\ plf$

$v_{L\_s} := \frac{V_{diaphL\_EL}}{L_T} = 130.58\ plf$

6" Nailing

$Check_w := \text{if } v_{L\_w} \le v_{w6} = \text{"OK"}$ "OK" else "NG!!"

$Check_s := \text{if } v_{L\_s} \le v_{s6} = \text{"OK"}$ "OK" else "NG!!"

Transverse Base Shear

$v_{T\_w} := \frac{V_{diaphT\_WL}}{L_L} = 110.47\ plf$

$v_{T\_s} := \frac{V_{diaphT\_EL}}{L_L} = 128.88\ plf$

6" Nailing

$Check_w := \text{if } v_{T\_w} \le v_{w6} = \text{"OK"}$ "OK" else "NG!!"

$Check_w := \text{if } v_{T\_s} \le v_{s6} = \text{"OK"}$ "OK" else "NG!!"

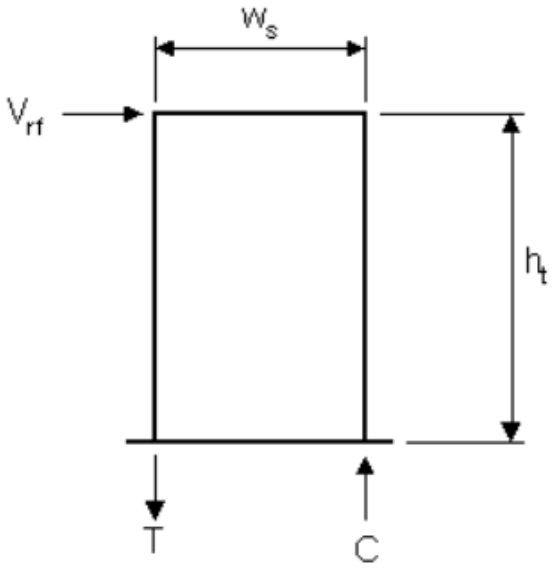
Use 19/32 APA Shtg w/ 10d nails @ 6"o.c. @ panel edges, 12" o.c. @ interior supports. Provide blocking @ panel edges.

S14

Upper Floor to Roof Analysis

Shear Wall Analysis - Upper Floor to Roof (ref. ANSI/AF&PA SDPWS -2021)

SWF WIND IN - PLANE SHEAR



$h_t := 8\ ft$ — Wall height

$L_s := 4\ ft + 11\ ft$ — Total shear wall length

$SDL_{rf} = 12\ psf$ — Dead load of roof

$R := R_{Lrf\_F} = 5.48\ kip$ — Reaction at grid

$w_{rf} := \frac{6\ ft}{2} + 1.5\ ft = 4.5\ ft$ — Tributary width of roof on wall

$p_{ext} = 12\ psf$ — Dead load of exterior walls

$w_s := 4\ ft$ — Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 2$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ — Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ — Shear load at top of wall (ASD) — $0.88\ kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD) — $M_{ot} = 7.01\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ — Roof load — $P_{rf} = 0.22\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ — Wall load — $P_w = 0.38\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ — Resisting moment (ASD) — $M_{res} = 0.72\ kip\ ft$

S15

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$w_v := \frac{V_{rf}}{w_s} = 219.06\ plf$ $check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0$ "NG" else "OK" $ratio := \frac{w_v}{w_{all}} = 0.60$ $check_{wv} = \text{"OK"}$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.1095\ kip$ Shear load to each anchor

$Check_a := \text{if } V_{sp} > Z_{ll}$ "NG" else "OK" $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.66$ $Check_a = \text{"OK"}$

Use 16d Nails @ 6" o.c. staggered

Holdown

$T := \frac{M_{ot} - M_{res}}{w_s} = 1.57\ kip$ $check_T := \text{if } T > 0.15\ kip$ "HD REQ'D" else "NOT REQ'D" $check_T = \text{"HD REQ'D"}$

$T_{all} := CS16 = 1705\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0$ "NG" else "OK" $ratio := \frac{T}{T_{all}} = 0.9223$ $check_{HD} = \text{"OK"}$

(E) Simpson CS16 each end OK, therefore no upgrades req'd

S16

SWF SEISMIC IN - PLANE SHEAR

$h_t := h_t = 8\ ft$ Wall height

$L_s := L_s = 15\ ft$ Total shear wall length

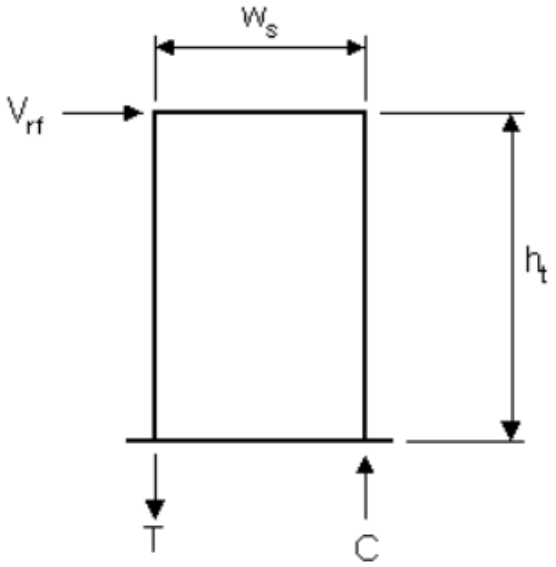
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lrf\_F\_EL} = 3.61\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 4.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 4\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 2$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $0.58\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD)

$M_{ot} = 4.62\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.22\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 0.38\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD)

$M_{res} = 0.72\ kip\ ft$

S17

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.7\ plf$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$w_v := \frac{V_{rf}}{w_s} = 144.31\ plf$ $check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0$ "NG" else "OK" $ratio := \frac{w_v}{w_{all}} = 0.55$ $check_{wv} = \text{"OK"}$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0722\ kip$ Shear load to each anchor

$Check_a := \text{if } V_{sp} > Z_{ll}$ "NG" else "OK" $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.44$ $Check_a = \text{"OK"}$

Use 16d Nails @ 6" o.c. staggered

Holdown

$T := \frac{M_{ot} - M_{res}}{w_s} = 0.97\ kip$ $check_T := \text{if } T > 0.150\ kip$ "HD REQ'D" else "NOT REQ'D" $check_T = \text{"HD REQ'D"}$

$T_{all} := CS16 = 1705\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0$ "NG" else "OK" $ratio := \frac{T}{T_{all}} = 0.5716$ $check_{HD} = \text{"OK"}$

(E) Simpson CS16 each end OK, therefore no upgrades req'd

S18

SWH WIND IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := 8.5\ ft$ Total shear wall length

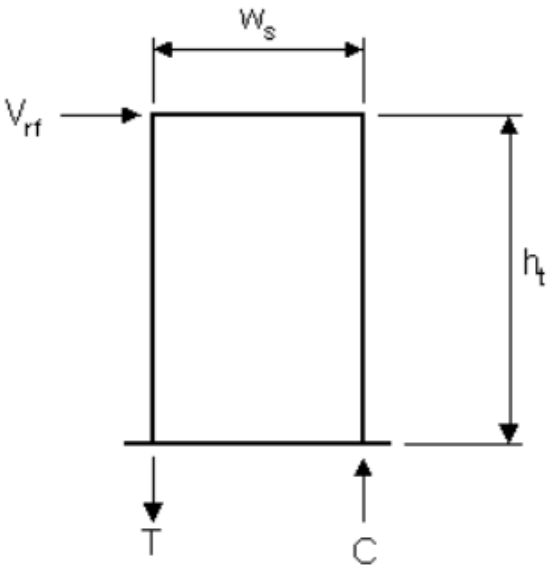
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lrf\_H} = 2.69\ kip$ Reaction at grid

$w_{rf} := \frac{2\ ft}{2} + 1.5\ ft = 2.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 8.5\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.9412$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $1.61\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD)

$M_{ot} = 12.91\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.25\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 0.82\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD)

$M_{res} = 2.73\ kip\ ft$

S19

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 189.9\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.52 \qquad check_{wv} = \text{"OK"}$$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0949\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.58 \qquad Check_a = \text{"OK"}$$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 1.2\ kip \qquad check_T := \text{if } T > 0.15\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := MST37 = 2705\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{T}{T_{all}} = 0.4428 \qquad check_{HD} = \text{"OK"}$$

Use Simpson MST37

S20

SWH SEISMIC IN - PLANE SHEAR

$h_t := h_t = 8\ ft$ — Wall height

$L_s := L_s = 8.5\ ft$ — Total shear wall length

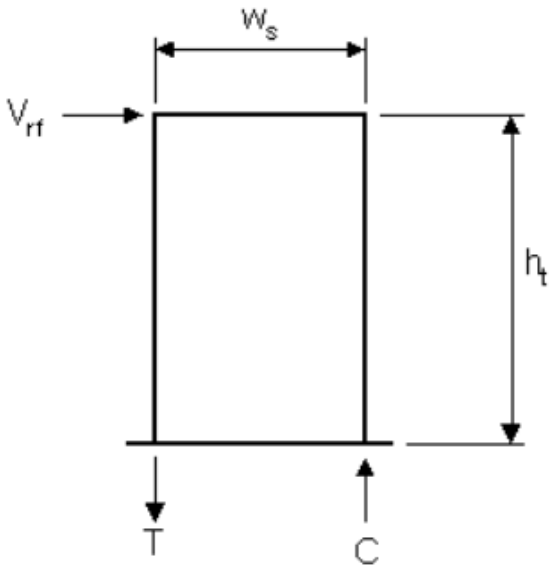
$SDL_{rf} = 12\ psf$ — Dead load of roof

$R := R_{Lrf\_H\_EL} = 1.77\ kip$ — Reaction at grid

$w_{rf} := w_{rf} = 2.5\ ft$ — Tributary width of roof on wall

$p_{ext} = 12\ psf$ — Dead load of exterior walls

$w_s := w_s = 8.5\ ft$ — Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.9412$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ — Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ — Shear load at top of wall (ASD)

$1.06\ kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD)

$M_{ot} = 8.51\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ — Roof load

$P_{rf} = 0.25\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ — Wall load

$P_w = 0.82\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ — Resisting moment (ASD)

$M_{res} = 2.73\ kip\ ft$

S21

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.7\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 125.1\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.48 \qquad check_{wv} = \text{"OK"}$$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $16d$ Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0626\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.38 \qquad Check_a = \text{"OK"}$$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.68\ kip \qquad check_T := \text{if } T > 0.150\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := MST37 = 2705\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{T}{T_{all}} = 0.2512 \qquad check_{HD} = \text{"OK"}$$

Use Simpson MST37

Holdown w/ Overstrength Factor $\Omega_{hd} := \Omega_o - 0.5 = 2.5$ (ref. table 12.2-1, footnote'b')

$$T := \frac{M_{ot} \cdot \Omega_{hd} - M_{res}}{w_s} = 2.18\ kip \qquad check_T := \text{if } T > 0.150\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

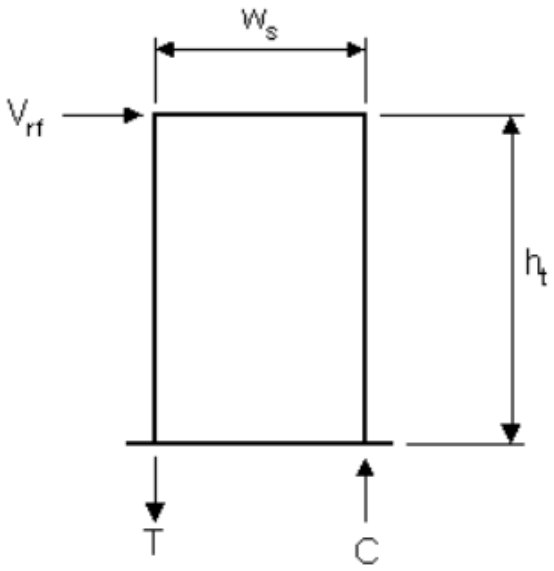
$T_{all} := MSTC48B3 = 3975\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Use Simpson MSTC48B3

S22

SW3 WIND IN - PLANE SHEAR



$h_t := 8\ ft$ Wall height

$L_s := 20.5\ ft$ Total shear wall length

$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Trf\_3} = 2.79\ kip$ Reaction at grid

$w_{rf} := \frac{6\ ft}{2} + 1.5\ ft = 4.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 20.5\ ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.3902$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $1.67\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 13.37\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 1.11\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 1.97\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 18.91\ kip\ ft$

S23

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} = 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 81.55\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.22$$

$check_{wv} = \text{"OK"}$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0408\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.25$$

$Check_a = \text{"OK"}$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -0.27\ kip \qquad check_T := \text{if } T > 0.15\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"NOT REQ'D"}$

S24

SW3 SEISMIC IN - PLANE SHEAR

$h_t := h_t = 8\ ft$ — Wall height

$L_s := L_s = 20.5\ ft$ — Total shear wall length

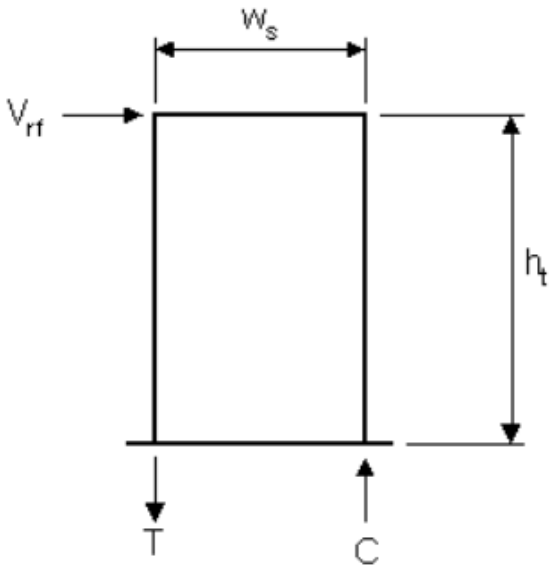
$SDL_{rf} = 12\ psf$ — Dead load of roof

$R := R_{Trf\_3\_EL} = 3.01\ kip$ — Reaction at grid

$w_{rf} := w_{rf} = 4.5\ ft$ — Tributary width of roof on wall

$p_{ext} = 12\ psf$ — Dead load of exterior walls

$w_s := w_s = 20.5\ ft$ — Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.3902$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ — Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ — Shear load at top of wall (ASD)

$1.8\ kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD)

$M_{ot} = 14.44\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ — Roof load

$P_{rf} = 1.11\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ — Wall load

$P_w = 1.97\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ — Resisting moment (ASD)

$M_{res} = 18.91\ kip\ ft$

S25

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.7\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 88.05\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.34 \qquad check_{wv} = \text{"OK"}$$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.044\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.27 \qquad Check_a = \text{"OK"}$$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -0.22\ kip \qquad check_T := \text{if } T > 0.150\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"NOT REQ'D"}$$

Holdown w/ Overstrength Factor

$\Omega_{hd} := \Omega_o - 0.5 = 2.5$ (ref. table 12.2-1, footnote'b')

$$T := \frac{M_{ot} \cdot \Omega_{hd} - M_{res}}{w_s} = 0.84\ kip \qquad check_T := \text{if } T > 0.150\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

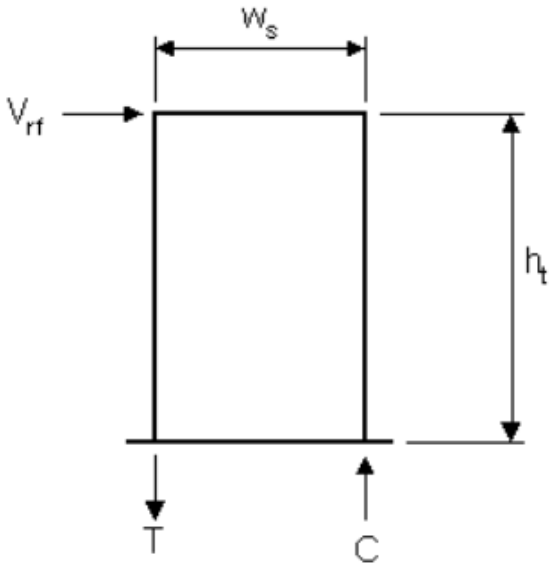
$T_{all} := MSTC48B3 = 3975\ lbf$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Use Simpson MSTC48B3

S26

SW5 WIND IN - PLANE SHEAR



$h_t := 8\ ft$ — Wall height

$L_s := 24.5\ ft$ — Total shear wall length

$SDL_{rf} = 12\ psf$ — Dead load of roof

$R := R_{Trf\_5} = 3.07\ kip$ — Reaction at grid

$w_{rf} := \frac{6\ ft}{2} + 1.5\ ft = 4.5\ ft$ — Tributary width of roof on wall

$p_{ext} = 12\ psf$ — Dead load of exterior walls

$w_s := 24.5\ ft$ — Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.3265$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ — Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ — Shear load at top of wall (ASD) — $1.84\ kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD) — $M_{ot} = 14.76\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ — Roof load — $P_{rf} = 1.32\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ — Wall load — $P_w = 2.35\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ — Resisting moment (ASD) — $M_{res} = 27.01\ kip\ ft$

S27

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 75.29\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.21$$

$check_{wv} = \text{"OK"}$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness 16d Nail $sp_a := 6\ in$ Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165\ kip$ Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0376\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.23$$

$Check_a = \text{"OK"}$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -0.5\ kip \qquad check_T := \text{if } T > 0.15\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"NOT REQ'D"}$

S28

SW5 SEISMIC IN - PLANE SHEAR

$h_t := h_t = 8\ ft$ Wall height

$L_s := L_s = 24.5\ ft$ Total shear wall length

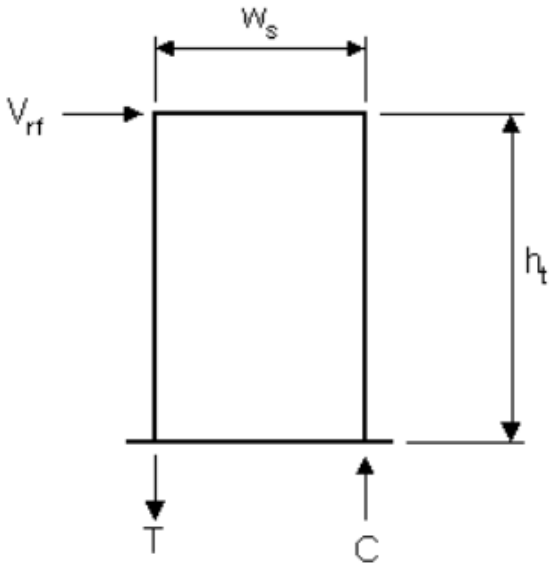
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Trf\_5\_EL} = 4.15\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 4.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 24.5\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.3265$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $2.49\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 19.92\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 1.32\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 2.35\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 27.01\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} = 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.7\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 101.62\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.39 \qquad check_{wv} = \text{"OK"}$$

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5$ in Sill plate thickness 16d Nail $sp_a := 6$ in Nail spacing

$Z_{ll} := 103\ lbf \cdot C_D = 0.165$ kip Allowable load parallel to grain (ref. NDS table 12N)

$V_{sp} := w_v \cdot sp_a = 0.0508$ kip Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.31 \qquad Check_a = \text{"OK"}$$

Use 16d Nails @ 6" o.c. staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -0.29 \text{ kip} \qquad check_T := \text{if } T > 0.150 \text{ kip "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"NOT REQ'D"}$$

Holdown w/ Overstrength Factor

$\Omega_{hd} := \Omega_o - 0.5 = 2.5$ (ref. table 12.2-1, footnote'b')

$$T := \frac{M_{ot} \cdot \Omega_{hd} - M_{res}}{w_s} = 0.93 \text{ kip} \qquad check_T := \text{if } T > 0.150 \text{ kip "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := MSTC48B3 = 3975$ lbf Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Use Simpson MSTC48B3

S30

HRS/ST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI

Strap Ties

Straps are designed to transfer tension loads in a wide variety of applications.

HRS — Heavy strap designed for installation on the edge of 2x members. The HRS416Z installs with Strong-Drive® SDS Heavy-Duty Connector screws.

HTP — Heavy tie plate designed for installation on the side of 2x4 or larger members.

LSTA and MSTA — Designed for use on the edge of 2x members, with a nailing pattern that reduces the potential for splitting.

LSTI and MSTI — Light and medium straps that are suitable where pneumatic-nailing is necessary through diaphragm decking and wood chord open-web trusses.

MST — High-capacity strap that can be installed with either nails or bolts. Suitable for double 2x member connections or greater.

MSTC — High-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

Finish: Galvanized. Some products are available in stainless steel, ZMAX® coating or black powder coat (add PC to sku); contact Simpson Strong-Tie. See Corrosion Information, pp. 13–15.

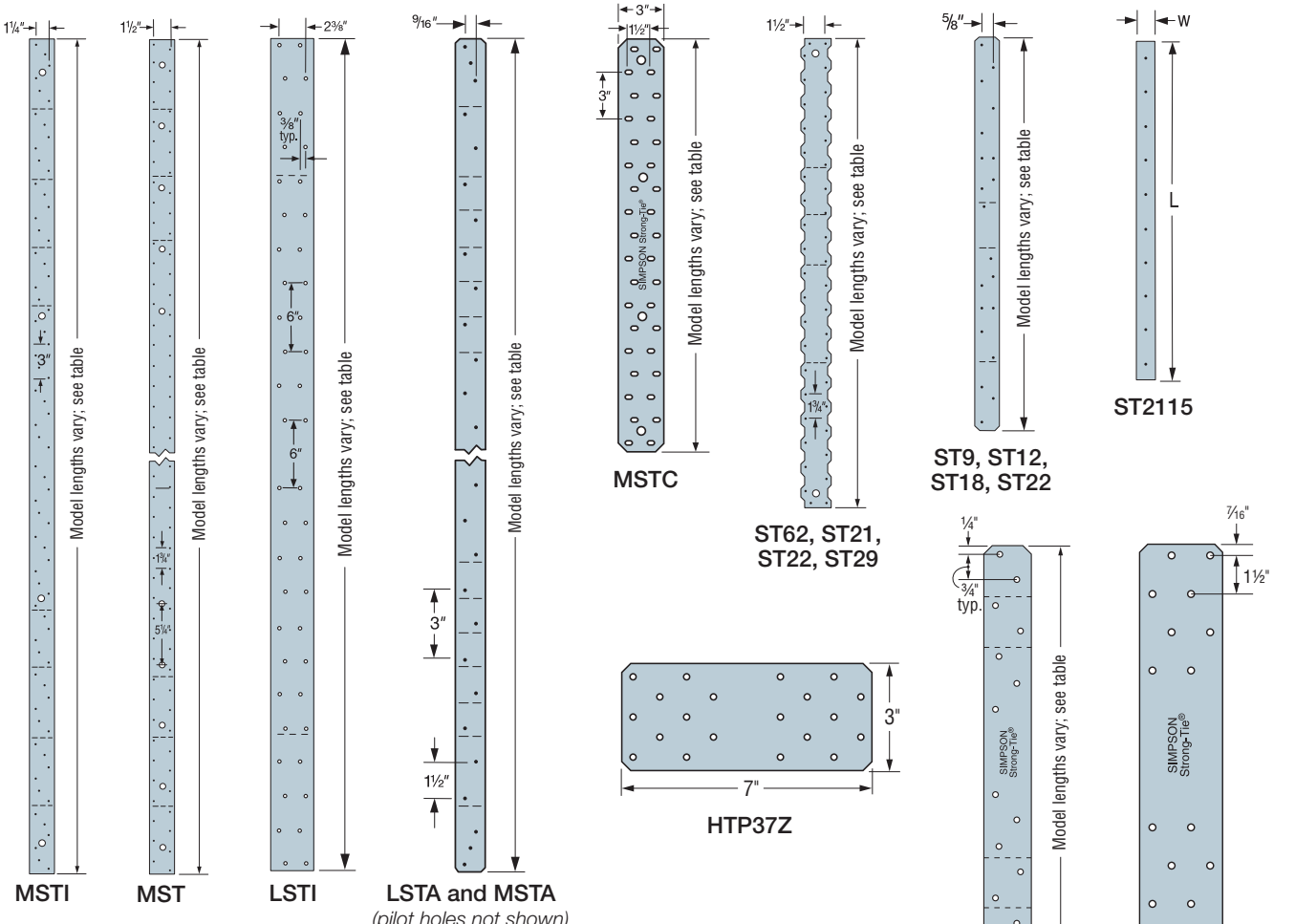
Installation: Use all specified fasteners; see General Notes

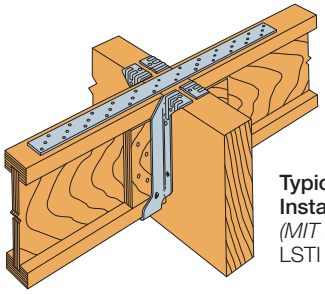
Options: Special sizes can be made to order; contact Simpson Strong-Tie

Codes: See p. 12 for Code Reference Key Chart

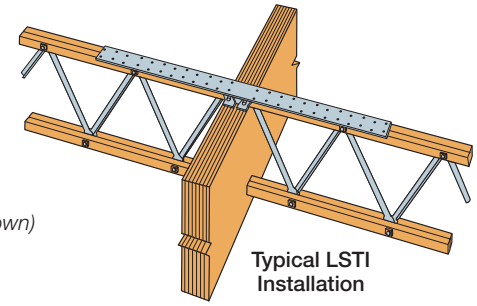
MSTC and RPS meet code requirements for reinforcing cut members (16 gauge) at top plate and RPS at sill plate.
International Residential Code® — 2012/2015/2018 R602.6.1
International Building Code® — 2012/2015/2018 2308.9.8

(For RPS, refer to p. 303.)

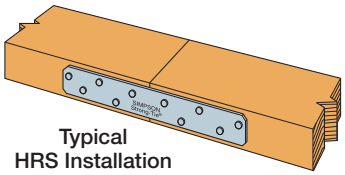




Typical MSTI Installation
(MIT hanger shown)
LSTI similar



Typical LSTI Installation



Typical HRS Installation

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Strap Ties (cont.)

Codes: See p. 12 for Code Reference Key Chart

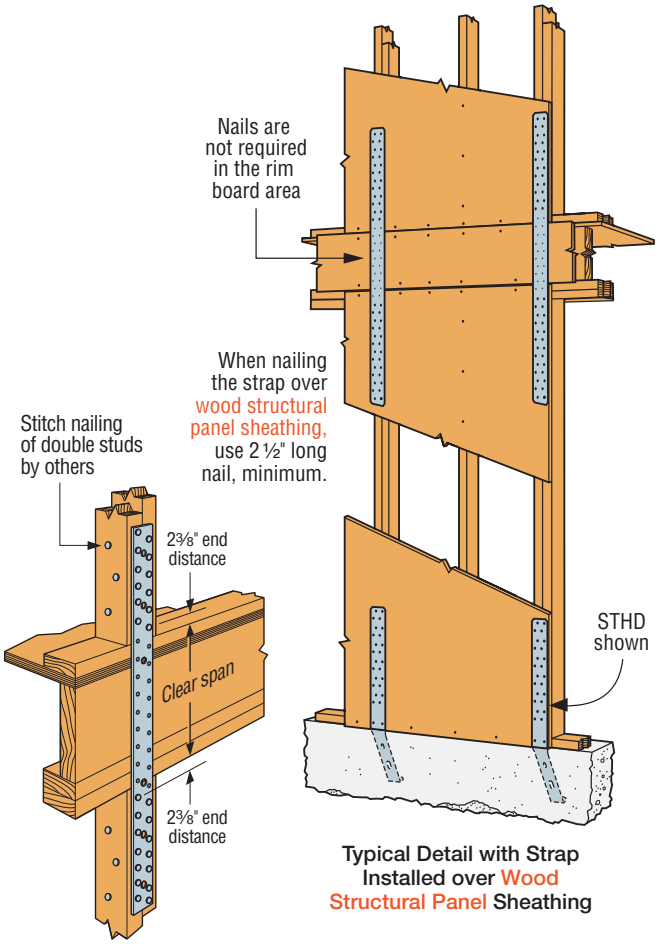
These products are available with additional corrosion protection. For more information, see p. 15.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Floor to Floor Span Table

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)
MSTA49	18	(26) 0.148 x 2½	2,020	2,020
	16	(26) 0.148 x 2½	2,020	2,020
MSTC28	18	(12) 0.148 x 3¼	1,150	995
	16	(16) 0.148 x 3¼	1,535	1,330
MSTC40	24	(20) 0.148 x 3¼	1,920	1,660
	18	(28) 0.148 x 3¼	2,690	2,325
	16	(32) 0.148 x 3¼	3,070	2,655
MSTC52	24	(36) 0.148 x 3¼	3,455	2,990
	18	(44) 0.148 x 3¼	4,225	3,650
	16	(48) 0.148 x 3¼	4,610	3,985
MSTC66	30	(48) 0.148 x 3¼	4,775	4,130
	24	(54) 0.148 x 3¼	5,375	4,645
	18	(64) 0.148 x 3¼	5,850	5,505
	16	(68) 0.148 x 3¼	5,850	5,850
MSTC78	30	(64) 0.148 x 3¼	5,850	5,505
	24	(72) 0.148 x 3¼	5,850	5,850
	18	(76) 0.148 x 3¼	5,850	5,850
MST37	24	(14) 0.162 x 2½	1,720	1,500
	18	(20) 0.162 x 2½	2,460	2,140
	16	(22) 0.162 x 2½	2,705	2,355
MST48	24	(26) 0.162 x 2½	3,210	2,780
	18	(32) 0.162 x 2½	3,950	3,425
	16	(34) 0.162 x 2½	4,200	3,640
MST60	30	(34) 0.162 x 2½	4,605	3,995
	24	(40) 0.162 x 2½	5,240	4,700
	18	(46) 0.162 x 2½	6,235	5,405
MST72	30	(48) 0.162 x 2½	6,505	5,640
	24	(54) 0.162 x 2½	6,730	6,345
	18	(62) 0.162 x 2½	6,730	6,475

See footnotes below.



Floor-to-Floor Tie Installation Showing a Clear Span

Typical Detail with Strap Installed over Wood Structural Panel Sheathing

Model No.	Ga.	Dimensions (in.) W	Dimensions (in.) L	Fasteners (Total) Nails (in.)	Fasteners (Total) Bolts Qty.	Fasteners (Total) Bolts Dia.	Allowable Tension Loads (DF/SP) Nails (160)	Allowable Tension Loads (DF/SP) Bolts (160)	Allowable Tension Loads (SPF/HF) Nails (160)	Allowable Tension Loads (SPF/HF) Bolts (160)	Code Ref.
MST27	12	2⅛6	27	(30) 0.162 x 2½	4	½	3,700	2,165	3,210	2,000	IBC, FL, LA
MST37		2⅛6	37½	(42) 0.162 x 2½	6	½	5,070	3,030	4,495	2,800	
MST48		2⅛6	48	(50) 0.162 x 2½	8	½	5,310	3,675	5,190	3,395	
MST60	10	2⅛6	60	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150	
MST72		2⅛6	72	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150	

1. See pp. 260–261 for Straps and Ties General Notes.
2. Install bolts or nails as specified by Designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and minimum member thickness: MST – 2½".
4. Splitting may be a problem with installations on lumber smaller than 3½"; either fill every nail hole with 0.148" x 1½" nails or fill every other hole with 0.162" x 2½" nails. Reduce the allowable load based on the size and quantity of fasteners used.
5. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

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Main to Upper FLoor Analysis

Shear Wall Analysis - Main Upper Floor (ref. ANSI/AF&PA SDPWS -2021)

SWF WIND IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := 23.67\ ft$ Total shear wall length

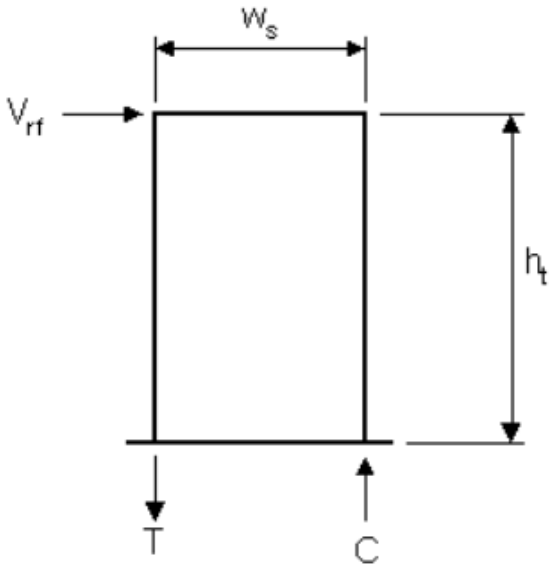
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_F} = 11.01\ kip$ Reaction at grid

$w_{rf} := \frac{13\ ft + 1.33\ ft}{2}$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 23.67\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.338$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $6.6\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 52.83\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 2.04\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 2.27\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 30.59\ kip\ ft$

S33

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 278.99\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.76$$

$check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 48\ in$ Anchor spacing

$Z_{ll} := v_{A.625\_2x} \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 1.116\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.75$$

$Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.9397\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"HD REQ'D"}$

$T_{all} := HD5A = 3.75\ kip$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"}$$

$check_{HD} = \text{"OK"}$

Existing HD5A OK, therefore no upgrades req'd

SWF SEISMIC IN - PLANE SHEAR

$h_t := 10\ ft$ Wall height

$L_s := L_s = 23.67\ ft$ Total shear wall length

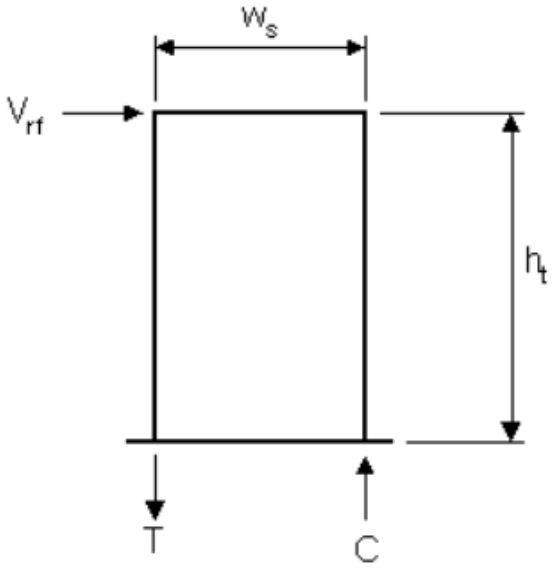
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_F\_EL} = 6.33\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 7.165\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 23.67\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.4225$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $3.8\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 37.98\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 2.04\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 2.84\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 34.62\ kip\ ft$

S35

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.71\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 160.45\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.62$$

$check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 48\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.6418\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.4313$$

$Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.1418\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"NOT REQ'D"}$

$T_{all} := HD5A = 3.75\ kip$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"}$$

$check_{HD} = \text{"OK"}$

Existing HD5A OK, therefore no upgrades req'd

S36

SWG WIND IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := 5.75\ ft + 4.75\ ft$ Total shear wall length

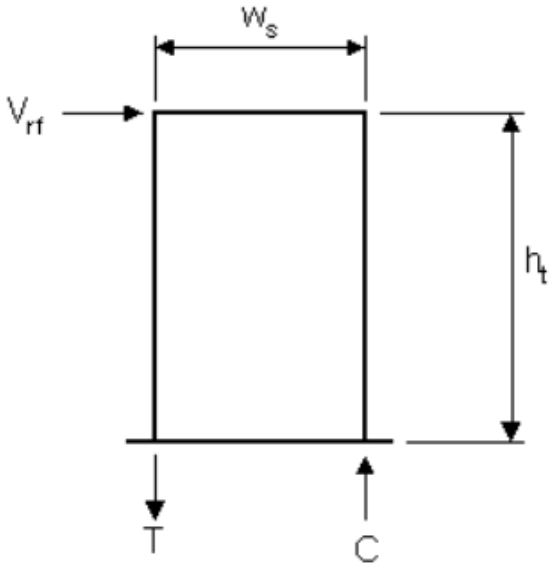
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_G} = 3.6\ kip$ Reaction at grid

$w_{rf} := \frac{13\ ft}{2} + 1.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 4.75\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 1.6842$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$
"NG"
else
"OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$
1.0
else
$1.25 - 0.125 \cdot \frac{h_t}{w_s}$

Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s} \right) \cdot 0.6$ Shear load at top of wall (ASD) $0.98\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 7.82\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.46\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 0.46\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2} \right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 1.3\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} = 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Double Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 205.76\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.56 \qquad check_{wv} = \text{"OK"}$$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 48\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.823\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.5531 \qquad Check_a = \text{"OK"}$$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 1.3725\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := DTT2Z = 2.145\ kip$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Use Simpson DTT2Z w/ 1/2" Dia anchor (8" embed into footing)

SWG SEISMIC IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := L_s = 10.5\ ft$ Total shear wall length

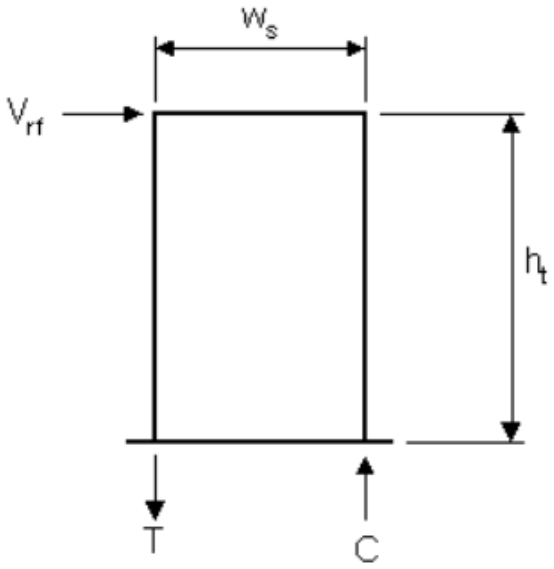
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_G\_EL} = 1.77\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 8\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 4.75\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 1.6842$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s} \right) \cdot 0.6$ Shear load at top of wall (ASD) $0.48\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 3.85\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.46\ kip$

$P_w := p_{ext} \cdot (h_t) \cdot w_s$ Wall load $P_w = 0.46\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2} \right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 1.3\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.71\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 101.27\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.39 \qquad check_{wv} = \text{"OK"}$$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 48\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.4051\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.2722 \qquad Check_a = \text{"OK"}$$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.5366\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := DTT2Z$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Use Simpson DTT2Z w/ 1/2" Dia anchor (8" embed into footing)

SWH WIND IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := 4.5\ ft$ Total shear wall length

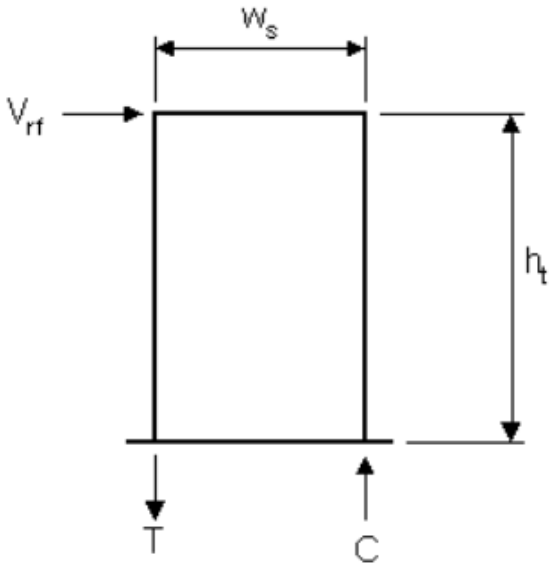
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_H} = 4.49\ kip$ Reaction at grid

$w_{rf} := \frac{5\ ft + 14\ ft}{2} + 1.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 4.5\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 1.7778$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$
"NG"
else
"OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$
1.0
else
$1.25 - 0.125 \cdot \frac{h_t}{w_s}$

Aspect ratio factor

$(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $2.69\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 21.55\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.59\ kip$

$P_w := p_{ext} \cdot \left(2 \cdot h_t\right) \cdot w_s$ Wall load $P_w = 0.86\ kip$

$M_{res} := \left(\left(P_{rf} + P_w\right) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 1.97\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{3\_8d} \cdot n}{\Omega_{D\_w}} = 685\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 3" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 598.74\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.87$$

$check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 16\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.7983\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.5365$$

$Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 16" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 4.3525\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"HD REQ'D"}$

$T_{all} := HDU4 = 4.565\ kip$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"}$$

$check_{HD} = \text{"OK"}$

Use Simpson HDU4 w/ 5/8" Dia anchor (12" embed into footing)

S42

SWH SEISMIC IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := L_s = 4.5\ ft$ Total shear wall length

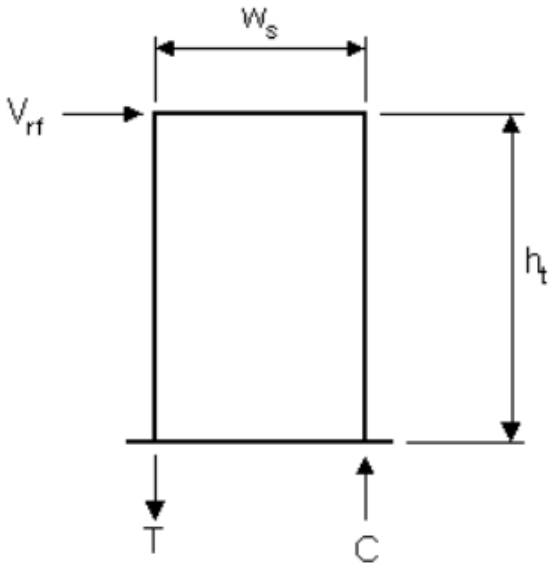
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Lup\_H\_EL} = 2.66\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 11\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 4.5\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 1.7778$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $1.6\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 12.76\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.59\ kip$

$P_w := p_{ext} \cdot (2 \cdot h_t) \cdot w_s$ Wall load $P_w = 0.86\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 1.97\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{4\_8d} \cdot n}{\Omega_{D\_s}} = 380.36\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 4" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 354.46\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.93$$

$check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 32\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.9452\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.6352$$

$Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 32" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 2.3983\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"HD REQ'D"}$

$T_{all} := HDU4$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"}$$

$check_{HD} = \text{"OK"}$

Use Simpson HDU4 w/ 5/8" Dia anchor (12" embed into footing)

S44

SW3 WIND IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := 14.33\ ft$ Total shear wall length

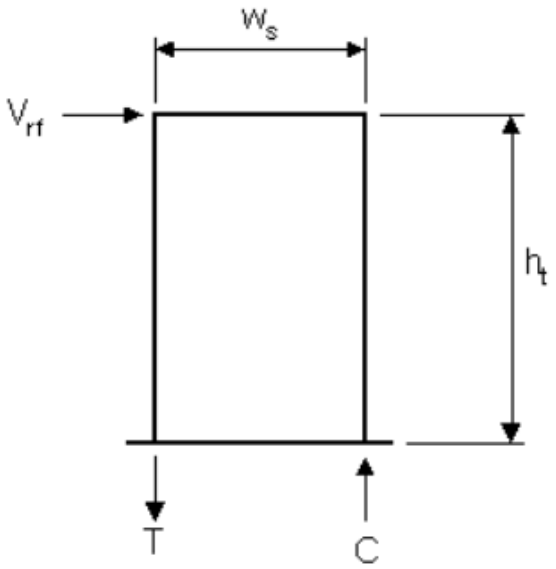
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Tup\_3} = 7.03\ kip$ Reaction at grid

$w_{rf} := \frac{8\ ft + 1.33\ ft}{2} + 1.5\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := 14.33\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.5583$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s} \right) \cdot 0.6$ Shear load at top of wall (ASD) $4.22\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 33.74\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 1.06\ kip$

$P_w := p_{ext} \cdot (2 \cdot h_t) \cdot w_s$ Wall load $P_w = 2.75\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2} \right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 16.39\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_w}} = 365\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 294.35\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.81 \qquad check_{wv} = \text{"OK"}$$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5$ in Sill plate thickness $dia_a := 0.625$ in Anchor dia $sp_a := 48$ in Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488$ kip Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 1.1774$ kip Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.7913 \qquad Check_a = \text{"OK"}$$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 1.2113\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := HD5A = 3.75$ kip Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Existing HD5A OK, therefore no upgrades req'd

S46

SW3 SEISMIC IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := L_s = 14.33\ ft$ Total shear wall length

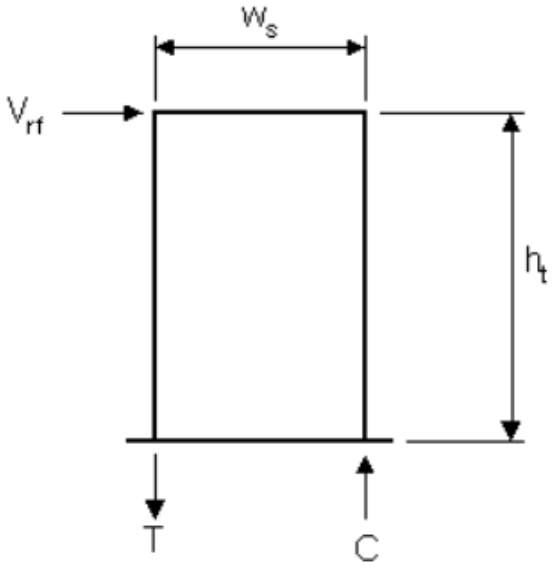
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Tup\_3\_EL} = 5.42\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 6.165\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 14.33\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.5583$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 1$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s} \right) \cdot 0.6$ Shear load at top of wall (ASD) $3.25\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 26.04\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 1.06\ kip$

$P_w := p_{ext} \cdot (2 \cdot h_t) \cdot w_s$ Wall load $P_w = 2.75\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2} \right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 16.39\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 1$ sides

$$w_{all} := \frac{(WSP) \cdot w_{6\_8d} \cdot n}{\Omega_{D\_s}} = 260.71\ plf$$

Single Sided 7/16" Plywood/OSB w/ 8d @ 6" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 227.13\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.87 \qquad check_{wv} = \text{"OK"}$$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 1.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 48\ in$ Anchor spacing

$Z_{ll} := 930\ lbf \cdot C_D = 1.488\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.9085\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.6106 \qquad Check_a = \text{"OK"}$$

Use 5/8" Dia. Anchor @ 48" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.6736\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"} \qquad check_T = \text{"HD REQ'D"}$$

$T_{all} := HDU4$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"} \qquad check_{HD} = \text{"OK"}$$

Existing HD5A OK, therefore no upgrades req'd

SW5 WIND IN - PLANE SHEAR

$h_t := 8\ ft$ — Wall height

$L_s := 3\ ft + 7\ ft \cdot \frac{w_{6\_8d}}{2 \cdot w_{3\_8d}}$ — Total shear wall length

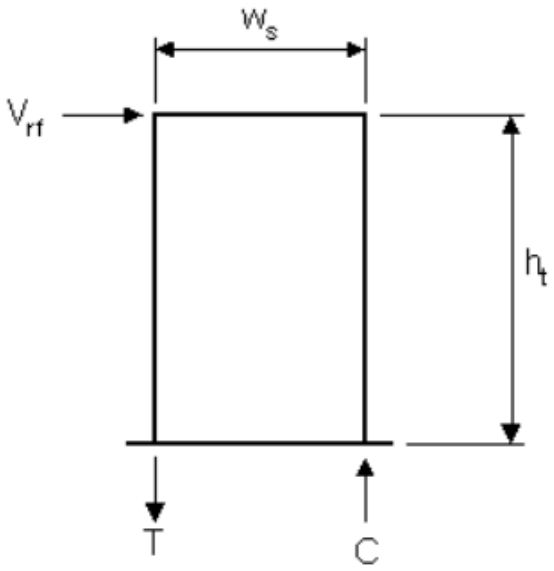
$SDL_{rf} = 12\ psf$ — Dead load of roof

$R := R_{Tup\_5} = 9.39\ kip$ — Reaction at grid

$w_{rf} := \frac{1.33\ ft + 6\ ft}{2} + 1.5\ ft$ — Tributary width of roof on wall

$p_{ext} = 12\ psf$ — Dead load of exterior walls

$w_s := 3\ ft$ — Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 2.6667$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ — Aspect ratio factor

$(WSP) = 0.9167$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s} \right) \cdot 0.6$ — Shear load at top of wall (ASD)

$3.47\ kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD)

$M_{ot} = 27.79\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ — Roof load

$P_{rf} = 0.19\ kip$

$P_w := p_{ext} \cdot (2 \cdot h_t) \cdot w_s$ — Wall load

$P_w = 0.58\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2} \right) \cdot 0.6$ — Resisting moment (ASD)

$M_{res} = 0.69\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_w} = 2.0$ (ref. section 4.3.3) $n := 2$ sides

$w_{all} := \frac{(WSP) \cdot w_{3\_8d} \cdot n}{\Omega_{D\_w}} = 1255.83\ plf$ **Double Sided 7/16" Plywood/OSB w/ 8d @ 3" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)**

$w_v := \frac{V_{rf}}{w_s} = 1158.06\ plf$ $check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0$ "NG" else "OK" $ratio := \frac{w_v}{w_{all}} = 0.92$ $check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 2.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 8\ in$ Anchor spacing

$Z_{ll} := 1180\ lbf \cdot C_D = 1.888\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.772\ kip$ Shear load to each anchor

$Check_a := \text{if } V_{sp} > Z_{ll}$ "NG" else "OK" $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.4089$ $Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 8" o.c. (6" min. embed)

Holdown

$T := \frac{M_{ot} - M_{res}}{w_s} = 9.0359\ kip$ $check_T := \text{if } T > 0.25\ kip$ "HD REQ'D" else "NOT REQ'D" $check_T = \text{"HD REQ'D"}$

$T_{all} := HDU11 = 9.535\ kip$ Allowable tension load (ref. Simpson Load Tables)

$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0$ "NG" else "OK" $check_{HD} = \text{"OK"}$

Use Simpson HDU11 w/ 1" Dia anchor (16" embed into footing)

S50

SW5 SEISMIC IN - PLANE SHEAR

$h_t := 8\ ft$ Wall height

$L_s := L_s = 4.865\ ft$ Total shear wall length

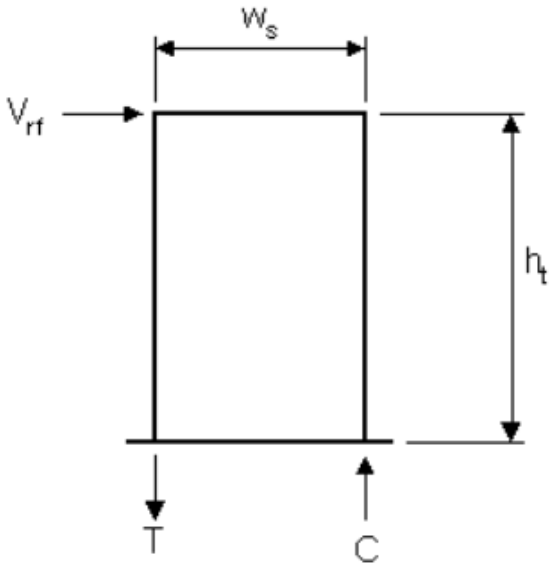
$SDL_{rf} = 12\ psf$ Dead load of roof

$R := R_{Tup\_5\_EL} = 6.48\ kip$ Reaction at grid

$w_{rf} := w_{rf} = 5.165\ ft$ Tributary width of roof on wall

$p_{ext} = 12\ psf$ Dead load of exterior walls

$w_s := w_s = 3\ ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 2.6667$

$check_{ratio} := \text{if } \frac{h_t}{w_s} > 3.5$ "NG" else "OK"

$check_{ratio} = \text{"OK"}$

$(WSP) := \text{if } \frac{h_t}{w_s} < 2.0$ 1.0 else $1.25 - 0.125 \cdot \frac{h_t}{w_s}$ Aspect ratio factor $(WSP) = 0.9167$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) \cdot 0.6$ Shear load at top of wall (ASD) $2.4\ kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 19.17\ kip\ ft$

Resisting Forces

$P_{rf} := SDL_{rf} \cdot w_{rf} \cdot w_s$ Roof load $P_{rf} = 0.19\ kip$

$P_w := p_{ext} \cdot (2 \cdot h_t) \cdot w_s$ Wall load $P_w = 0.58\ kip$

$M_{res} := \left((P_{rf} + P_w) \cdot \frac{w_s}{2}\right) \cdot 0.6$ Resisting moment (ASD) $M_{res} = 0.69\ kip\ ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS) $\Omega_{D\_s} := 2.8$ (ref. section 4.3.3) $n := 2$ sides

$$w_{all} := \frac{(WSP) \cdot w_{3\_8d} \cdot n}{\Omega_{D\_s}} = 897.02\ plf$$

Double Sided 7/16" Plywood/OSB w/ 8d @ 3" O.C. Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A, 15/32 values)

$$w_v := \frac{V_{rf}}{w_s} = 798.86\ plf \qquad check_{wv} := \text{if } \frac{w_v}{w_{all}} > 1.0 \text{ "NG" else "OK"} \qquad ratio := \frac{w_v}{w_{all}} = 0.89$$

$check_{wv} = \text{"OK"}$

Sill Plate Anchorage $C_D := 1.6$

$t_{sp} := 2.5\ in$ Sill plate thickness $dia_a := 0.625\ in$ Anchor dia $sp_a := 8\ in$ Anchor spacing

$Z_{ll} := 1180\ lbf \cdot C_D = 1.888\ kip$ Allowable load parallel to grain (ref. NDS table 12E)

$V_{sp} := w_v \cdot sp_a = 0.5326\ kip$ Shear load to each anchor

$$Check_a := \text{if } V_{sp} > Z_{ll} \text{ "NG" else "OK"} \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.2821$$

$Check_a = \text{"OK"}$

Use 5/8" Dia. Anchor @ 8" o.c. (6" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 6.1623\ kip \qquad check_T := \text{if } T > 0.25\ kip \text{ "HD REQ'D" else "NOT REQ'D"}$$

$check_T = \text{"HD REQ'D"}$

$T_{all} := HDU8$ Allowable tension load (ref. Simpson Load Tables)

$$check_{HD} := \text{if } \frac{T}{T_{all}} > 1.0 \text{ "NG" else "OK"}$$

$check_{HD} = \text{"OK"}$

Use Simpson HDU8 w/ 7/8" Dia anchor (14" embed into footing)

S52

HDU/DTT

Holdowns

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.

HDU holdowns are pre-deflected during the manufacturing process, virtually eliminating deflection under load due to material stretch. They use Strong-Drive® SDS Heavy-Duty Connector screws which install easily, reduce fastener slip and provide a greater net section when compared to bolts.

The DTT tension ties are designed for lighter-duty holdown applications on single 2x posts. The DTT1Z is installed with nails or Strong-Drive SD Connector screws and the DTT2Z installs easily with the Strong-Drive SDS Heavy-Duty Connector screws (included). The DTT1Z holdowns have been tested for use in designed shearwalls and prescriptive braced wall panels as well as prescriptive wood-deck applications (see p. 289 for deck applications).

For more information on holdown options, contact Simpson Strong-Tie.

HDU Features:

- Uses Strong-Drive SDS Heavy-Duty Connector screws which install easily, reduce fastener slip and provide a greater net section area of the post compared to bolts
- Strong-Drive SDS Heavy-Duty Connector screws are supplied with the holdowns to ensure proper fasteners are used
- No stud bolts to countersink at openings

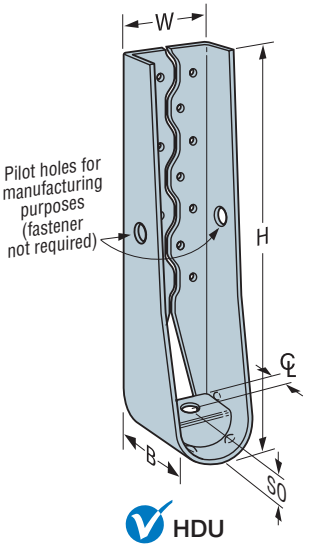
Material: See table

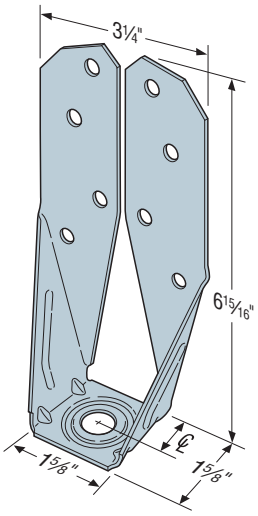
Finish: HDU — galvanized; DTT1Z and DTT2Z — ZMAX® coating; DTT2SS — stainless steel

Installation:

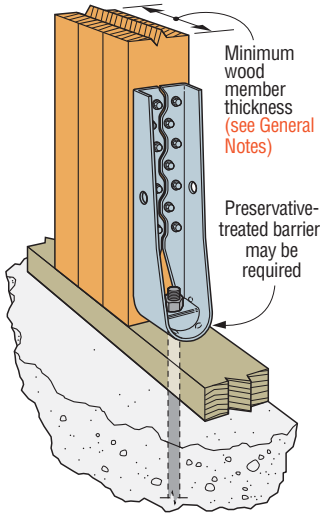
- See Holdown and Tension Tie General Notes on pp. 49–50.
- The HDU requires no additional washer; the DTT requires a standard-cut washer (included with DTT2Z) be installed between the nut and the seat.
- Strong-Drive SDS Heavy-Duty Connector screws install best with a low-speed high-torque drill with a ⅜" hex-head driver.
- Fasteners and crescent washer are included with the holdowns. For replacements, order part no. SDS25212-HDU_. (Fill in the size needed, e.g. HDU2.)

Codes: See p. 12 for Code Reference Key Chart

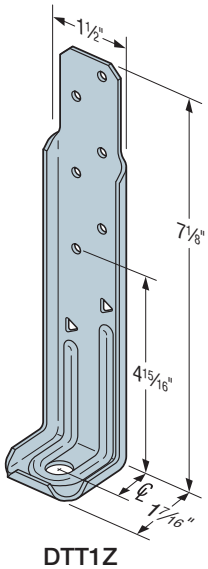




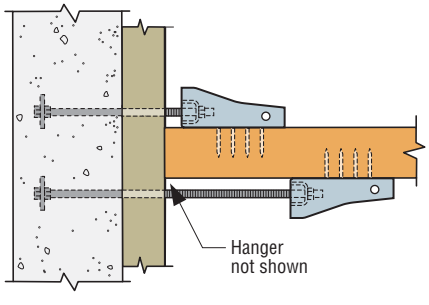
DTT2Z
U.S. Patent 8,555,580



Vertical HDU Installation



DTT1Z
U.S. Patent Pending



Horizontal HDU Offset Installation
(plan view)
See Holdown and Tension Tie General Notes.

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HDU/DTT

Holdowns (cont.)

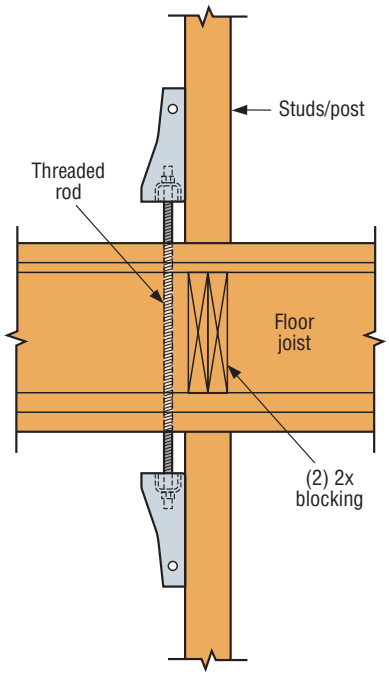
These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335-337 for more information.

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)			Code Ref.
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT1Z	14	1½	7⅛	1⁷⁄₁₆	¾	⅜	⅜	(6) SD #9 x 1½	1½ x 5½	840	840	0.17	IBC, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
SS DTT2Z	14	3¼	6¹⁵⁄₁₆	1⅝	¹³⁄₁₆	⅜	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
SS DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8¹⁄₁₆	3¼	1⁵⁄₁₆	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10¹⁵⁄₁₆	3¼	1⁵⁄₁₆	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13³⁄₁₆	3¼	1⁵⁄₁₆	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,335	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25¹¹⁄₁₆	3½	1⁹⁄₁₆	1⁹⁄₁₆	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.



Typical HDU Tie Between Floors

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Roof Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Roof Joists	Passed (67% M)	1 piece(s) 2 x 8 DF No.2 @ 24" OC	
Hip/Valley Beams	Passed (102% M)	2 piece(s) 2 x 10 DF No.2	
Ridge Beam	Passed (82% ΔT)	1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam	
Header @ Ridge Beam	Passed (79% M)	2 piece(s) 2 x 10 DF No.2	
Upper Floor Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Floor Joists	Passed (84% M)	1 piece(s) 2 x 10 DF No.2 @ 16" OC	
Beam Below Bearing Wall	Passed (84% ΔT)	1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam	
Beam @ Grid G	Passed (85% ΔL)	1 piece(s) 5 1/2" x 16 1/2" 24F-V4 DF Glulam	
Headers Perp to Joists	Passed (53% M)	2 piece(s) 2 x 6 DF No.2	
Main Floor Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Floor: Joist	Passed (84% M)	1 piece(s) 2 x 10 DF No.2 @ 16" OC	

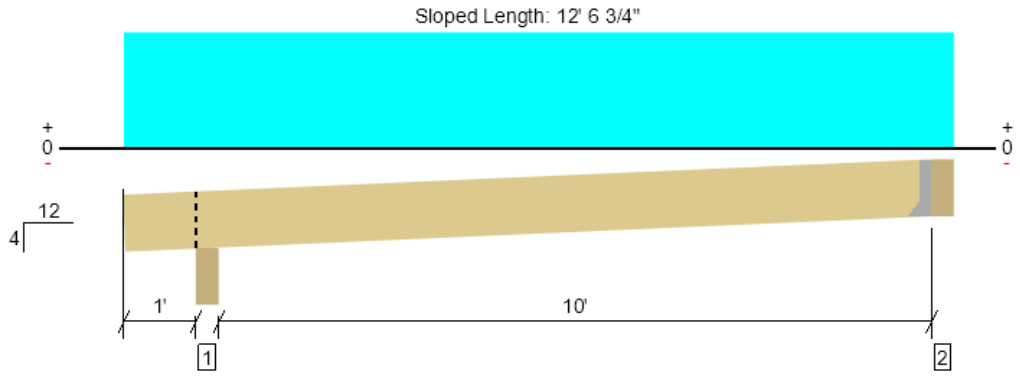
FORTEWEB

MEMBER REPORT

PASSED

Roof Framing, Roof Joists

1 piece(s) 2 x 8 DF No.2 @ 24" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	413 @ 11' 5 1/2"	1406 (1.50")	Passed (29%)	--	1.0 D + 1.0 S (Alt Spans)
Shear (lbs)	367 @ 10' 10 5/8"	1501	Passed (24%)	1.15	1.0 D + 1.0 S (Alt Spans)
Moment (Ft-lbs)	1046 @ 6' 4 3/4"	1564	Passed (67%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.176 @ 6' 4 1/4"	0.539	Passed (L/733)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.286 @ 6' 4 5/16"	0.719	Passed (L/452)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 12' 3 3/8"
System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 4/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Beveled Plate - DF	5.50"	5.50"	1.50"	203	257	321	524	Blocking
2 - Hanger on 7 1/4" DF beam	5.50"	Hanger[1]	1.50"	173	221	277	450	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 4" o/c	
Bottom Edge (Lu)	12' 1" o/c	

- Maximum allowable bracing intervals based on applied load.
- Dimensions for lateral bracing intervals are measured along the length of the member for sloped conditions.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LRU26Z	1.94"	N/A	4-10dx1.5	5-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 11' 11"	24"	15.0	20.0	25.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Allen Rishel NKH Engineering (206) 641-1733 allen@nkhengineering.com	

Weyerhaeuser

8/1/2024 9:13:12 PM UTC
ForteWEB v3.8, Engine: V8.4.1.22, Data: V8.1.6.2
File Name: 24-065 4227

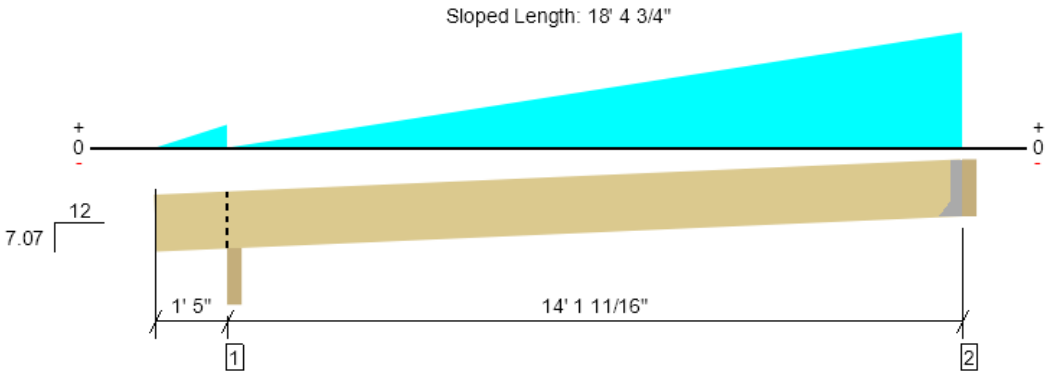
FORTEWEB

MEMBER REPORT

PASSED

Roof Framing, Hip/Valley Beams

2 piece(s) 2 x 10 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1525 @ 15' 6 11/16"	2813 (1.50")	Passed (54%)	--	1.0 D + 1.0 S (Alt Spans)
Shear (lbs)	1316 @ 14' 10 11/16"	3830	Passed (34%)	1.15	1.0 D + 1.0 S (Alt Spans)
Moment (Ft-lbs)	4159 @ 9' 7 1/16"	4059	Passed (102%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.328 @ 8' 9 3/4"	0.812	Passed (L/594)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.610 @ 8' 9 5/8"	1.083	Passed (L/320)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 18' 6 1/8"
System : Roof
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 7.07/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - SPF	3.50"	3.50"	1.50"	416	447	864	Blocking
2 - Hanger on 9 1/4" SPF beam	3.50"	Hanger[1]	1.50"	696	828	1525	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6" o/c	
Bottom Edge (Lu)	18' 1" o/c	

- Maximum allowable bracing intervals based on applied load.
- Dimensions for lateral bracing intervals are measured along the length of the member for sloped conditions.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LSSR210-2Z	1.88"	N/A	22-16dx2.5	18-16dx2.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 15' 6 11/16"	N/A	7.0	--	
1 - Tapered (PLF)	0 to 1' 5"	N/A	0.0 to 23.8	0.0 to 35.4	Generated from Roof Geometry
2 - Tapered (PLF)	1' 5" to 15' 6 11/16"	N/A	0.0 to 117.8	0.0 to 176.8	Generated from Roof Geometry

- Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Allen Rishel NKH Engineering (206) 641-1733 allen@nkhengineering.com	

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

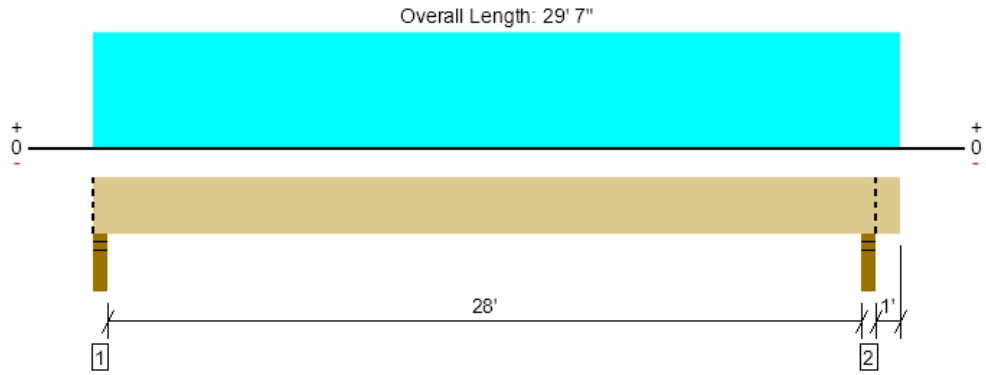
FORTEWEB

MEMBER REPORT

PASSED

Roof Framing, Ridge Beam

1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3337 @ 28' 5 1/4"	12031 (3.50")	Passed (28%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2810 @ 27' 2"	15085	Passed (19%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	21733 @ 14' 3 7/16"	36607	Passed (59%)	1.15	1.0 D + 1.0 S (Alt Spans)
Neg Moment (Ft-lbs)	-143 @ 28' 5 1/4"	29619	Passed (0%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.883 @ 14' 3 9/16"	1.414	Passed (L/384)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	1.540 @ 14' 3 9/16"	1.885	Passed (L/220)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 29' 7"
System : Roof
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.95 that was calculated using length L = 28' 2 7/8".
- Critical negative moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 1' 2 5/16".
- 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	Roof Live	Snow	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	1329	1429	1786	3115	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	1424	1530	1913	3337	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	29' 7" o/c	
Bottom Edge (Lu)	29' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 29' 7"	N/A	18.0	--	--	
1 - Uniform (PSF)	0 to 29' 7" (Front)	5'	15.0	20.0	25.0	Default Load

• Side loads are assumed to not induce cross-grain tension.

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ForteWEB Software Operator	Job Notes
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File Name: 24-065 4227

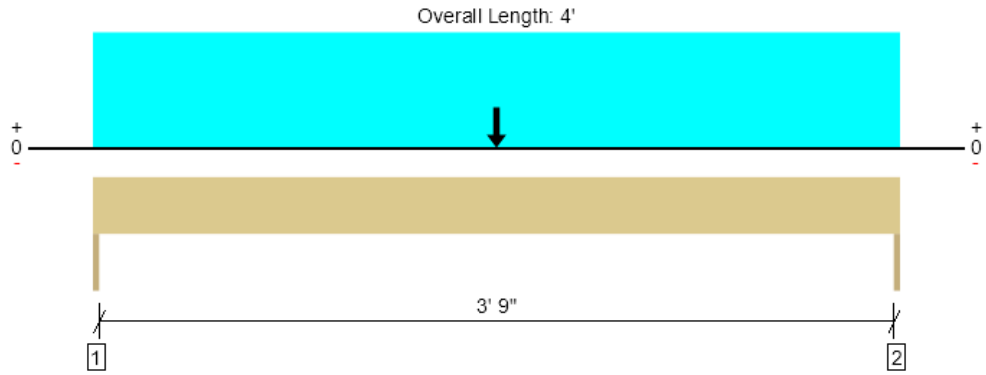
FORTEWEB

MEMBER REPORT

PASSED

Roof Framing, Header @ Ridge Beam

2 piece(s) 2 x 10 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1652 @ 0	2813 (1.50")	Passed (59%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1609 @ 10 3/4"	3830	Passed (42%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3209 @ 2'	4059	Passed (79%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.013 @ 2'	0.100	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.024 @ 2'	0.200	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 4'
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	709	715	943	1652	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	709	715	943	1652	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	4' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 4'	N/A	7.0	--	--	
1 - Uniform (PSF)	0 to 4'	1'	15.0	-	25.0	Default Load
2 - Point (lb)	2'	N/A	1329	1429	1786	Linked from: Ridge Beam, Support 1

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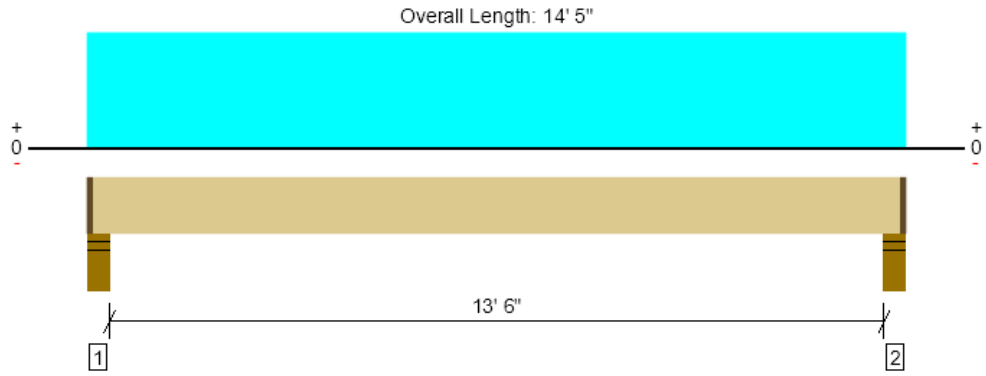
FORTEWEB

MEMBER REPORT

PASSED

Upper Floor Framing, Floor Joists

1 piece(s) 2 x 10 DF No.2 @ 16" OC



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	519 @ 4 1/2"	3750 (4.00")	Passed (14%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	438 @ 1' 2 3/4"	1665	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1712 @ 7' 2 1/2"	2029	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.264 @ 7' 2 1/2"	0.342	Passed (L/620)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.364 @ 7' 2 1/2"	0.683	Passed (L/451)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 14' 2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	5.50"	4.00"	1.50"	144	384	529	1 1/2" Rim Board
2 - Stud wall - DF	5.50"	4.00"	1.50"	144	384	529	1 1/2" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 8" o/c	
Bottom Edge (Lu)	14' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 14' 5"	16"	15.0	40.0	Default Load

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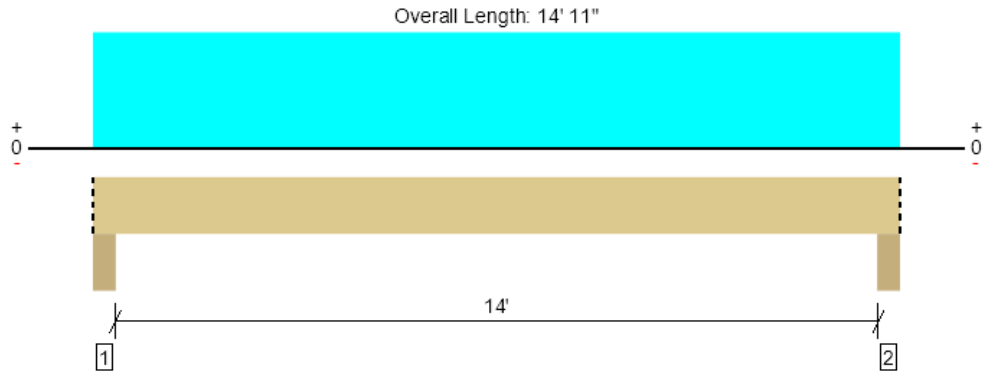
MEMBER REPORT

PASSED

Upper Floor Framing, Beam Below Bearing Wall

1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1847 @ 4"	12513 (5.50")	Passed (15%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1548 @ 1' 2 1/2"	6400	Passed (24%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	6286 @ 7' 5 1/2"	10868	Passed (58%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.364 @ 7' 5 1/2"	0.475	Passed (L/470)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.600 @ 7' 5 1/2"	0.712	Passed (L/285)	--	1.0 D + 1.0 S (All Spans)

Member Length : 14' 11"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 14' 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	Factored	
1 - Column - DF	5.50"	5.50"	1.50"	728	1119	1847	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	728	1119	1847	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	14' 11" o/c	
Bottom Edge (Lu)	14' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 14' 11"	N/A	7.7	--	
1 - Uniform (PSF)	0 to 14' 11" (Front)	6'	15.0	25.0	Default Load

• Side loads are assumed to not induce cross-grain tension.

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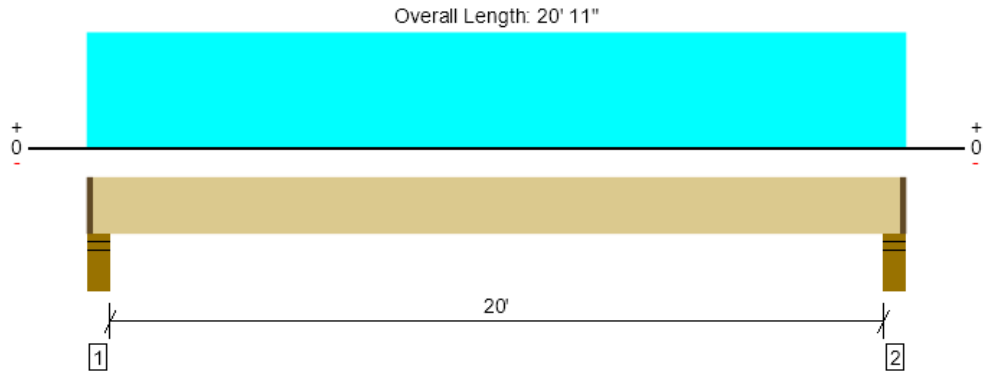
MEMBER REPORT

PASSED

Upper Floor Framing, Beam @ Grid G

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	8185 @ 4"	13750 (4.00")	Passed (60%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	6831 @ 1' 10"	16033	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	40599 @ 10' 5 1/2"	48183	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.572 @ 10' 5 1/2"	0.675	Passed (L/425)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.809 @ 10' 5 1/2"	1.013	Passed (L/301)	--	1.0 D + 1.0 L (All Spans)

Member Length : 20' 8"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.97 that was calculated using length L = 20' 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	Floor Live	Factored	
1 - Stud wall - DF	5.50"	4.00"	2.38"	2424	5857	8281	1 1/2" Rim Board
2 - Stud wall - DF	5.50"	4.00"	2.38"	2424	5857	8281	1 1/2" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	20' 8" o/c	
Bottom Edge (Lu)	20' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/2" to 20' 9 1/2"	N/A	22.1	--	
1 - Uniform (PSF)	0 to 20' 11" (Front)	14'	15.0	40.0	Default Load

• Side loads are assumed to not induce cross-grain tension.

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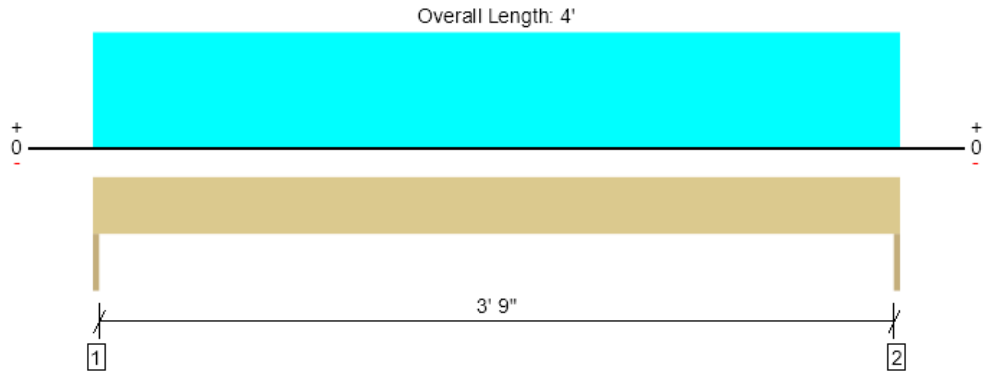
FORTEWEB

MEMBER REPORT

PASSED

Upper Floor Framing, Headers Perp to Joists

2 piece(s) 2 x 6 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	778 @ 0	2813 (1.50")	Passed (28%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	551 @ 7"	1980	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	778 @ 2'	1475	Passed (53%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.024 @ 2'	0.100	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.034 @ 2'	0.200	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 4'
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	218	560	778	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	218	560	778	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	4' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 4'	N/A	4.2	--	
1 - Uniform (PSF)	0 to 4'	7'	15.0	40.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Allen Rishel NKH Engineering (206) 641-1733 allen@nkhengineering.com	

Weyerhaeuser

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File Name: 24-065 4227

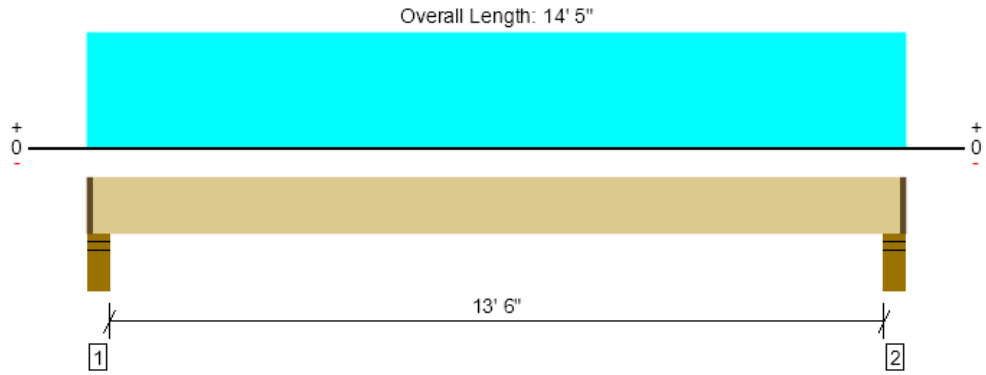
FORTEWEB

MEMBER REPORT

PASSED

Main Floor Framing, Floor: Joist

1 piece(s) 2 x 10 DF No.2 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	519 @ 4 1/2"	3750 (4.00")	Passed (14%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	438 @ 1' 2 3/4"	1665	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1712 @ 7' 2 1/2"	2029	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.264 @ 7' 2 1/2"	0.342	Passed (L/620)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.364 @ 7' 2 1/2"	0.683	Passed (L/451)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 14' 2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	5.50"	4.00"	1.50"	144	384	529	1 1/2" Rim Board
2 - Stud wall - DF	5.50"	4.00"	1.50"	144	384	529	1 1/2" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 8" o/c	
Bottom Edge (Lu)	14' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 14' 5"	16"	15.0	40.0	Default Load

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