Gass Apartment

Project Number: 20-172

4471 Tolt Ave

Carnation, WA 98014

Reviewed For Code Compliance

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09/28/2023

Structural Calculations

9/27/23

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PROJECT: Gass Apartments

DESIGNER: NKH & AKR

DATE: January 25th, 2021

JOB #: 20-172

PROJECT SUMMARY & DESIGN CRITERIA

Background

Project Summary:

This is a new two story, wood framed apartment for the Gass Family in Carnation, WA. The structure consists of wood roof & floor trusses/joists bearing on wood framed walls, posts, & beams. The building is supported by new concrete stem walls & shallow spread footings. This project is designed in accordance with the 2015 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). (2015). "National Design Specifications for Wood Construction (NDS)."
- American Wood Council (AWC). (2015). "Special Design Provisions for Wind and Seismic (SDWS)."
- American Concrete Institute (ACI). (2014). "Building Code Requirements for Structural Concrete (ACI 318-14)."
- American Institute of Steel Construction (AISC). (2011). "Steel Construction Manual." 14th Ed.
- American Society of Civil Engineers (ASCE). (2010). "Minimum Design Loads for Buildings and Other Structures."
- State of Washington (2015). "International Building Code (IBC)."
- Applied Technology Council (ATC). (2018). "Hazards by Location" https://hazards.atcouncil.org

Material Properties

Soil:

- Soil Bearing Pressure (min per IBC1806.2) $p_{brg} := 1500psf$
- Frost Depth $FD := 12in$
- Active & Passive Soil Pressure $q_a := 35 \cdot pcf$ $q_p := 250 \cdot pcf$

Concrete:

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

Steel:

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

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		$R := 1.5 \cdot psf$
Insulation		$I := 2.0 \cdot psf$
Ceiling		$C := 2 \cdot psf$
Sheathing	$t := 0.5in$	$SH := \left(\frac{t}{.125in}\right) \cdot 0.4 \cdot psf = 1.6 \cdot psf$
Structural Members		$S := 2.5 \cdot psf$
Lights		$L := 1 \cdot psf$
Mechanical		$M := 1.5 \cdot psf$
Misc.		$MISC := 2.9 \cdot psf$

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

Seismic Roof Dead Load

$SDL_{rf} := DL_{rf} - MISC = 12.1\,psf$ $\quad SDL_{rf} = 12\,psf$

Floor Dead Load

Flooring		$F := 1.5 \cdot psf$
Insulation		$I := 2.0 \cdot psf$
Ceiling		$C := 0 \cdot psf$
Sheathing	$t := 0.75in$	$SH := \left(\frac{t}{.125in}\right) \cdot 0.4 \cdot psf = 2.4 \cdot psf$
Structural Members		$S := 3.4 \cdot psf$
Lights		$L := 1 \cdot psf$
Mechanical		$M := 1.5 \cdot psf$
Misc.		$MISC := 3.2 \cdot psf$

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

Seismic Roof Dead Load

$SDL_{flr} := DL_{flr} = 15\,psf$ $\quad SDL_{flr} = 15 \cdot psf$

Wall Dead Loads

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

Live Loads

Roof	$LL_{rf} := 20 \cdot psf$	Roof Snow Load	$SL := 25psf$
Floor Live Load (Office)	$LL_{flr} := 50\,psf$		
Deck Live Load	$LL_{deck} := 1.5 \cdot LL_{flr} = 60\,psf$		

Deflection Criteria

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

NKH ENGINEERING

Lateral Analysis

References

Lateral Summary

General

LRFD

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

$L := 54\text{ft}$	Building Length	$SDL_{rf} = 12\cdot\text{psf}$	Seismic Roof Dead Load
$B := 25\text{ft}$	Building Width	$SDL_{flr} = 15\cdot\text{psf}$	Seismic Floor Dead Load
$h_{rf} := 24\text{ft}$	Avg Roof Height	$p_{ext\_w} = 10\cdot\text{psf}$	Exterior Stud Wall Load
$h_p := 0\text{ft}$	Parapet Height	$p_{int} = 9\cdot\text{psf}$	Interior Stud Wall Load
$h_{wall} := 10\text{ft}$	Wall Height	$a := \min(10\%\cdot B, 0.4h_{rf}) = 2.5\text{ ft}$	Width of Pressure Coefficient Zone

MWFRS (per ASCE 7-10, Chapter 26 & 27)

$\theta := \text{atan}\left(\frac{0\text{in}}{12\text{in}}\right) = 0\cdot\text{deg}$ Roof Slope

Design Velocity Pressure - Enclosed/Partially Enlosed Buildings

$V_w := 110$ mph	Basic Wind Speed (ref. figure 26.5-1A)
$K_d := 0.85$	Directionality Factor (ref. section 26.6)
$exp := \text{"B"}$	Exposure Category (ref. section 26.7)
$K_{zt} := 1.0$	Topographic Factor (ref. section 26.8)
$K_z = 0.66$	Velocity Pressure Exposure Coefficient (ref. table 27.3-1)

$q_z := 0.00256\cdot K_z\cdot K_{zt}\cdot K_d\cdot V_w^{\,2}\cdot(\text{psf})$ Velocity pressure (eq 27.3-1) $q_z = 17.4\cdot\text{psf}$

Design Wind Pressure

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

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

Walls

$GC_{pi} := \begin{pmatrix} -0.85 \\ 0.85 \end{pmatrix}$ Internal Pressure Coefficient (ref. table 26.11-1)

Veolcity Pressure Evaluated at Mean Roof Height, h

$q_h := q_z = 17.38 \cdot psf$

External Pressure Coefficients for Walls (ref. figure 27.4-1)

$\frac{L}{B} = 2.16$ $C_{pww} := 0.8$ Windward Wall $C_{plw} = -0.3$ Leeward Wall

Design MWFRS Wind Pressures (eq 27.4-1)

$$p_w := \max\left[p_{w\_min}, \max\left[q_h \cdot \left[G_e \cdot \left(C_{pww} + C_{plw}\right) - GC_{pi}\right]\right]\right] = 22.2 \cdot psf$$ $p_w = 22.2 \cdot psf$

Parapet (ref. section 27.4.5)

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

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

$p_p := \text{if}\left[h_p \le 0, 0psf, q_z \cdot \left(GC_{pnw} - GC_{pnL}\right)\right]$ Combined Net Pressure on Parapet $p_p = 0 \cdot psf$

Design Wind Pressure (cont'd)

$\frac{h_{rf}}{L} = 0.44$

Roof (fig. 27.4-1)

$GC_{pi} = \begin{pmatrix} -0.85 \\ 0.85 \end{pmatrix}$ Internal pressure coefficient (ref. table 26.11-1)

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

$\frac{h_{rf}}{L} = 0.44$ $C_{prf} := \begin{pmatrix} -0.9 \\ -0.18 \end{pmatrix}$ Windward & leeward coefficients

Veolcity pressure evaluated at mean roof height, h

$q_h := q_z = 17.4 \cdot psf$

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

$p_{rf1} := q_h \cdot \left(G_e \cdot \min\left(C_{prf}\right) - GC_{pi}\right) = \begin{pmatrix} 1.5 \\ -28.1 \end{pmatrix} \cdot psf$ $p_{rf2} := q_h \cdot \left(G_e \cdot \max\left(C_{prf}\right) - GC_{pi}\right) = \begin{pmatrix} 12.1 \\ -17.4 \end{pmatrix} \cdot psf$

$p_{rf} := \max\left(\left|\min\left(p_{rf1}\right)\right|, \left|\max\left(p_{rf2}\right)\right|\right) = 28.06 \cdot psf$ $p_{rf\_horiz} := p_{rf} \cdot \sin(\theta) = 0 \cdot psf$

$p_{w\_up} := 0.6DL_{rf} + 0.6 \cdot \left(\min\left(p_{rf1}, p_{rf2}\right)\right)$ Net uplift pressure (ASD) $p_{w\_up} = -7.8 \cdot psf$

Roof Overhangs

$C_{poh} := -0.8$ External pressure coefficients for roof overhangs (ref. 27.4.4)

$p_{oh} := q_z \cdot \left(G_e \cdot C_{poh}\right) + \min\left(p_{rf1}, p_{rf2}\right)$ Overhang pressure $p_{oh} = -39.9 \cdot psf$

$OH_{net} := 0.6DL_{rf} + 0.6 \cdot p_{oh}$ Net uplift pressure (ASD) $OH_{net} = -15 \cdot psf$

C&C (per ASCE 7-10, Chapter 30)

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

$GC_{pw4} := \begin{pmatrix} 1.0 \\ -1.1 \end{pmatrix}$ exterior pressure coefficients

$GC_{pw5} := \begin{pmatrix} 1.0 \\ -1.4 \end{pmatrix}$ exterior pressure coefficients (corner zone)

$p_{cc\_w4pos} := q_h \cdot (\max(GC_{pw4}) - GC_{pi}) = \begin{pmatrix} 32.1 \\ 2.6 \end{pmatrix} \cdot psf$ Positive design wind pressure (ref. eq. 30.4-1)

$p_{cc\_w4neg} := q_h \cdot (\min(GC_{pw4}) - GC_{pi}) = \begin{pmatrix} -4.3 \\ -33.9 \end{pmatrix} \cdot psf$ Negative design wind pressure

$p_{cc\_w5pos} := q_h \cdot (\max(GC_{pw5}) - GC_{pi}) = \begin{pmatrix} 32.1 \\ 2.6 \end{pmatrix} \cdot psf$ Corner zone positive design wind pressure

$p_{cc\_w5pos} := q_h \cdot (\min(GC_{pw5}) - GC_{pi}) = \begin{pmatrix} -9.6 \\ -39.1 \end{pmatrix} \cdot psf$ Corner zone negative design wind pressure

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

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

$p_{cc\_r1} := q_h \cdot (GC_{pr1} - GC_{pi}) = \begin{pmatrix} -0.9 \\ -30.4 \end{pmatrix} \cdot psf$

$p_{cc\_r2} := q_h \cdot (GC_{pr2} - GC_{pi}) = \begin{pmatrix} -14.8 \\ -44.3 \end{pmatrix} \cdot psf$

$p_{cc\_r3} := q_h \cdot (GC_{pr3} - GC_{pi}) = \begin{pmatrix} -30.4 \\ -60 \end{pmatrix} \cdot psf$

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

$p_{cc\_rpos} := q_h \cdot (GC_{pr\_pos} - GC_{pi}) = \begin{pmatrix} 23.5 \\ -6.1 \end{pmatrix} \cdot psf$

Wind Base Shear

$A_{wall\_L} := 1172ft^2$ $A_{roof\_L} := 0ft^2$ $A_{wall\_T} := 646ft^2$ $A_{roof\_T} := 0ft^2$

$V_{wu\_L} := p_w \cdot A_{wall\_L} + A_{roof\_L} p_{rf\_horiz}$ $V_{wu\_L} = 26 \cdot kip$ Longitudinal diaphragm shear

$V_{wu\_T} := p_w \cdot A_{wall\_T} + A_{roof\_T} p_{rf\_horiz}$ $V_{wu\_T} = 14.3 \cdot kip$ Transverse diaphragm shear

Seismic Main Floor - Roof (per ASCE 7-10, 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)

$S_{DS} := 0.804$ Design spectral acceleration parameter (ref. ATC summary report)

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

$\Omega_o := 2.5$ System overstrength factor (ref. table 12.2-1)

$C_d := 3.25$ Deflection amp. factor (ref. table 12.2-1)

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

$C_s := \dfrac{S_{DS}}{\left(\dfrac{R}{I_s}\right)} = 0.12$ Seismic response coefficient

$S_{D1} := 0.46$ $S_1 := 0.443$ < 0.6g therefore 12.8-6 does not apply

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

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

$T_a := C_t \cdot \left(\dfrac{h_n}{ft}\right)^x = 0.22$ EQ 12.8-7

$C_{smax} := \dfrac{S_{D1}}{T_a \cdot \left(\dfrac{R}{I_s}\right)}$ $C_{smax} = 0.33$

$C_s := \max\left(\min\left(C_s, C_{smax}\right), 0.01\right)$ $C_s = 0.124$

$C_{s\_min} := \dfrac{0.5 \cdot S_{D1}}{\left(\dfrac{R}{I_s}\right)} = 0.04$ EQ 12.8-6

$C_{s\_wood} := \rho \cdot C_s$ $C_{s\_wood} = 0.124$

Seismic Base Shear

Building Weights Contributing to Seismic Forces

Diaphragms

$W_{diaphragm} := 1266ft^2 \cdot SDL_{rf} + \left(1331ft^2\right) \cdot SDL_{flr}$ $\qquad W_{diaphragm} = 35 \cdot kip$

Walls

$W_{walls\_T} := \left(p_{ext\_w} + p_{int}\right) A_{wall\_T} \cdot 2$ $\qquad W_{walls\_T} = 25 \cdot kip$

$W_{walls\_L} := \left(p_{ext\_w} + p_{int}\right) A_{wall\_L} \cdot 2$ $\qquad W_{walls\_L} = 45 \cdot kip$

Shear Loads

$V_{su\_T} := C_{s\_wood} \cdot \left(W_{diaphragm} + W_{walls\_T}\right)$ $\qquad V_{su\_T} = 7.4 \cdot kip$

$V_{su\_L} := C_{s\_wood} \cdot \left(W_{diaphragm} + W_{walls\_L}\right)$ $\qquad V_{su\_L} = 9.87 \cdot kip$

Lateral Summary (ASD)

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

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

	Wind	*Seismic*
Transverse	$V_{w\_T} := 0.6V_{wu\_T} \cdot C_{sw\_cap} = 6.12 \cdot kip$	$V_{s\_T} := 0.7V_{su\_T} = 5.18 \cdot kip$

$V_T := if\left(V_{w\_T} > V_{s\_T}, \text{"WIND CONTROLS"}, \text{"SEISMIC CONTROLS"}\right) = \text{"WIND CONTROLS"}$

	Wind	*Seismic*
Longitudinal	$V_{w\_L} := 0.6V_{wu\_L} \cdot C_{sw\_cap} = 11.1 \cdot kip$	$V_{s\_L} := 0.7V_{su\_L} = 6.91 \cdot kip$

$V_L := if\left(V_{w\_L} > V_{s\_L}, \text{"WIND CONTROLS"}, \text{"SEISMIC CONTROLS"}\right) = \text{"WIND CONTROLS"}$

Lateral Forces - Roof

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

$h_{rf\_proj} := 0ft$ Roof Projection Above Wall

$p_w = 22.2 \cdot psf$ Design Wall Wind Pressure (ref. Wind Loading)

$p_{rf\_horiz} = 0 \cdot psf$ Design Roof Wind Pressure (ref. Wind Loading)

Longitudinal Wall Line Reactions (Ref. Shear Wall Diagram)

Reaction 1

$trib1 := \frac{23.25ft}{2} = 11.63\,ft$

$$R_{Lrf\_1} := \left[p_w \cdot \left(\frac{h_{wall}}{2}\right) + p_{rf\_horiz} \cdot h_{rf\_proj}\right] \cdot trib1 \qquad R_{Lrf\_1} = 1.29 \cdot kip$$

Reaction 2

$trib2 := \frac{27.75ft}{2} = 13.87\,ft$

$$R_{Lrf\_2} := \left[p_w \cdot \left(\frac{h_{wall}}{2}\right) + p_{rf\_horiz} \cdot h_{rf\_proj}\right] \cdot (trib1 + trib2) \qquad R_{Lrf\_2} = 2.82 \cdot kip$$

Reaction 3

$trib3 := trib2 = 13.87\,ft$

$$R_{Lrf\_3} := \left[p_w \cdot \left(\frac{h_{wall}}{2}\right) + p_{rf\_horiz} \cdot h_{rf\_proj}\right] \cdot (trib3) \qquad R_{Lrf\_3} = 1.54 \cdot kip$$

Transverse Wall Line Reactions (Ref. Shear Wall Diagram)

Reaction A

$tribA := \frac{25ft}{2} = 12.5\,ft$

$$R_{Trf\_A} := \left[p_w \cdot \left(\frac{h_{wall}}{2}\right) + p_{rf\_horiz} \cdot h_{rf\_proj}\right] \cdot tribA \qquad R_{Trf\_A} = 1.38 \cdot kip$$

Lateral Forces - Upper Floor Roof

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

$p_w = 22.2 \cdot psf$ Design Wall Wind Pressure (ref. Wind Loading)

Longitudinal Wall Line Reactions (Ref. Shear Wall Diagram)

Reaction 1 $trib1 := \frac{23.25ft}{2} = 11.63\,ft$

$R_{Lup\_1} := \left[p_w \cdot \left(h_{wall}\right)\right] \cdot trib1 + R_{Lrf\_1}$ $R_{Lup\_1} = 3.86 \cdot kip$

Reaction 2 $trib2 := \frac{22ft}{2} = 11\,ft$

$R_{Lup\_2} := \left[p_w \cdot \left(h_{wall}\right)\right] \cdot (trib1 + trib2) + R_{Lrf\_2}$ $R_{Lup\_2} = 7.84 \cdot kip$

Reaction 3 $trib3 := trib2 = 11\,ft$

$R_{Lup\_3} := \left[p_w \cdot \left(h_{wall}\right)\right] \cdot trib3 + p_w \cdot \frac{h_{wall}}{2} 6ft + R_{Lrf\_3}$ $R_{Lup\_3} = 4.64 \cdot kip$

Transverse Wall Line Reactions (Ref. Shear Wall Diagram)

Reaction A $tribA := \frac{21.5ft}{2} = 10.75\,ft$

$R_{Tup\_A} := \left[p_w \cdot \left(h_{wall}\right)\right] \cdot tribA + R_{Trf\_A}$ $R_{Tup\_A} = 3.77 \cdot kip$

Reaction B $tribB := tribA = 10.75\,ft$

$R_{Tup\_B} := \left[p_w \cdot \left(h_{wall}\right)\right] \cdot tribB + \left(p_w \cdot \frac{h_{wall}}{2}\right) 3.5ft + R_{Trf\_A}$ $R_{Tup\_B} = 4.15 \cdot kip$

Lateral Summary

Diaphragm Check

Diaphragm Check (ref. ANSI/AF&PA SDPWS-2015)

Aspect Ratio

$L_T := 21ft$ $\quad$ $L_L := 51ft$ $\quad$ Length & width of diaphragm

$check_D := if\left(\frac{L_L}{L_T} > 4, "NG", "OK"\right)$ $\quad$ $ratio := \frac{L_L}{L_T} = 2.43$ $\quad$ $check_D = "OK"$

Diaphragm Shear

Shear capacities for 15/32" APA Rated OSB/Plywood Sheathing - Un-Blocked (ref. table 4.2A):

$\Omega_D := 2.0$ $\quad$ ASD reduction factor

$v_{w6\_ub} := 600plf \div \Omega_D = 300 \cdot plf$ $\quad$ Allowable Wind Shear Capacity- 10d's @ 6" oc

Diaphragm

$L_T = 21\ ft$ $\quad$ Diaphragm length in transverse direction

$V_{diaphT} := R_{Tup\_B} \cdot 0.6 = 2.5 \cdot kip$ $\quad$ Diaphragm shear transverse direction

$L_L = 51\ ft$ $\quad$ Diaphragm length in transverse direction

$V_{diaphL} := R_{Lup\_2} \cdot 0.6 = 4.7 \cdot kip$ $\quad$ Diaphragm shear transverse direction

Transverse Shear

$v_T := \frac{V_{diaphT}}{L_L}$ $\quad$ Diaphragm shear

6" Nailing

$v := v_T = 49 \cdot plf$

$Check := if\left(v \le v_{w6\_ub}, "OK", "NG!!"\right)$ $\quad$ $Check = "OK"$ $\quad$ Use 6" nailing everywhere

Longitudinal Shear

$v_T := \frac{V_{diaphL}}{L_T}$ $\quad$ Diaphragm shear

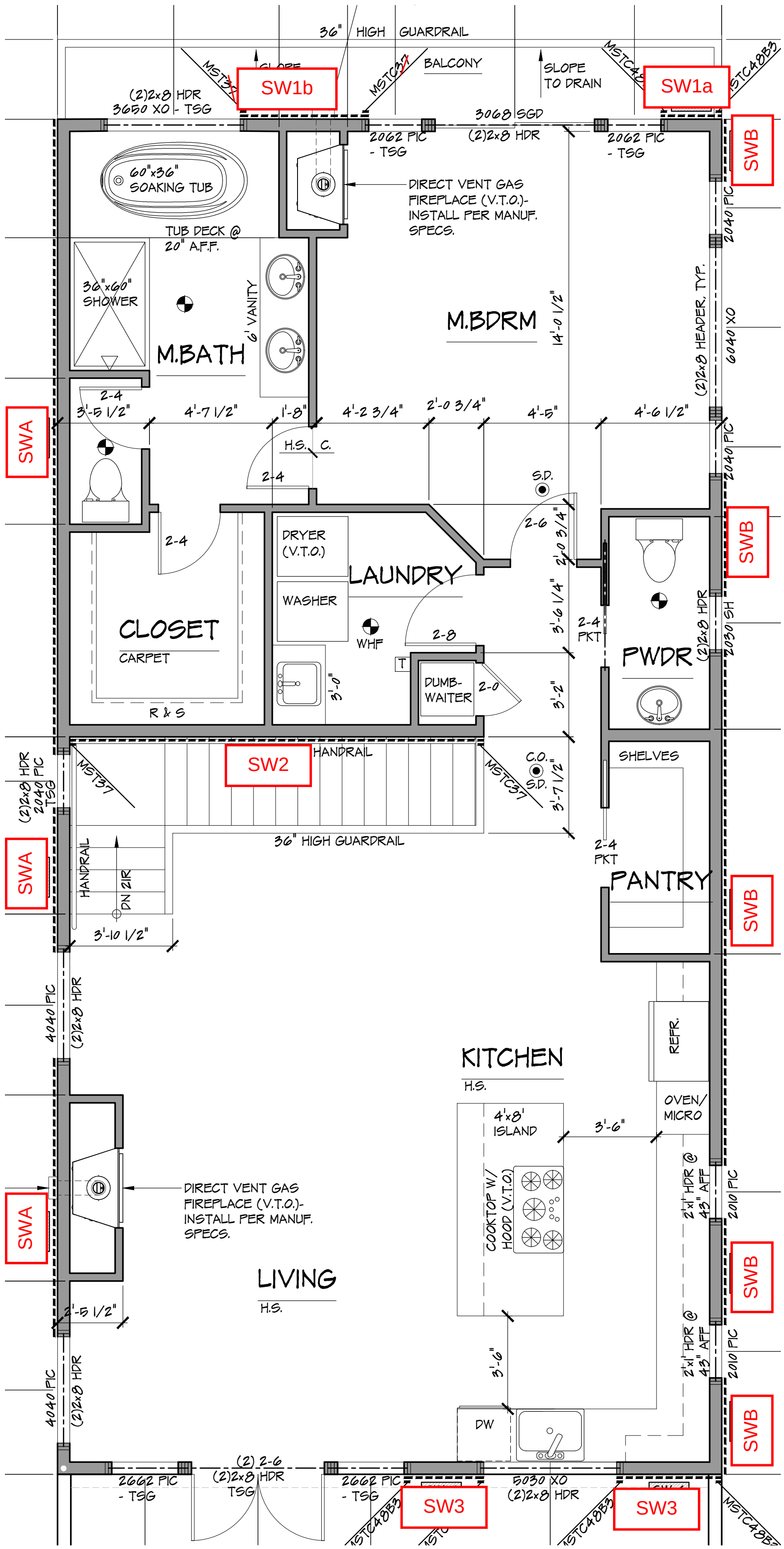
6" Nailing

$v := v_T = 224 \cdot plf$

$Check := if\left(v \le v_{w6\_ub}, "OK", "NG!!"\right)$ $\quad$ $Check = "OK"$ $\quad$ Use 6" nailing everywhere

Use 15/32 APA Shtg w/ 10d nails @ 6"o.c. @ panel edges,
12" o.c. @ interior supports.

Diaphragm Check



PROJECT: Gass Apartment

DESCRIPTION: Upper Floor Shearwall Keyplan

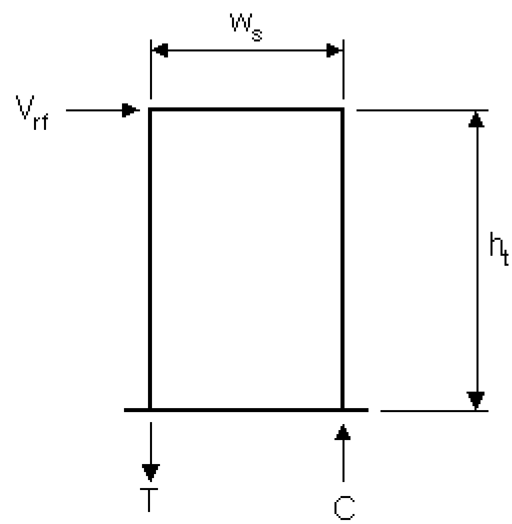
BY: AKR DATE: 1/25/2021 JOB #: 20-172

S12

Upper Floor Shear Walls

Shear Wall Check - Upper Floor to Roof (ref. ANSI/AF&PA SDPWS-2015)

SW1a IN - PLANE SHEAR



$h_t := 8 \cdot ft$ Wall height

$L_s := 4.75ft + 2.5ft$ Total shear wall length

$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Lrf\_1} = 1.29 \cdot kip$ Reaction at wall line

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

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 2.5ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 3.2$ $check_{ratio} := if\left(\frac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $check_{ratio} = "OK"$

$(WSP) := if\left(\frac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \frac{h_t}{w_s}\right)$ Aspect ratio factor $(WSP) = 0.9$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $V_{rf} = 0.27 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 2.1 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ Roof load $P_{rf} = 0.04 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $P_w = 0.2 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \frac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $M_{res} = 0.18 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 107 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 369.8 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right) \qquad check_{wv} = "OK"$$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.053 \cdot kip$ Shear load to each anchor

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.24 \qquad Check_a = "OK"$$

Use 16d Nail at 6"o.c. Staggered

Holdown

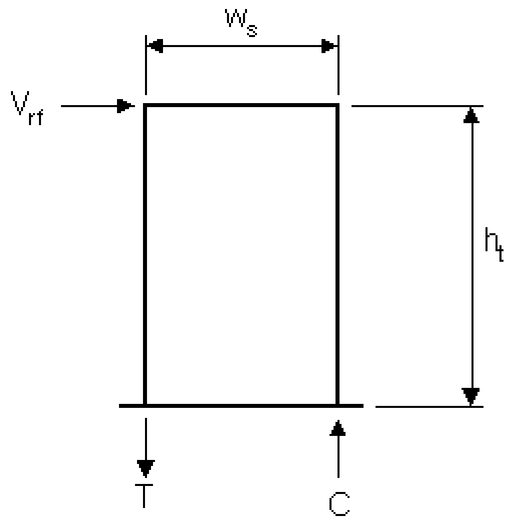
$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.78 \cdot kip \qquad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D") \qquad check_T = "HD REQ'D"$$

$T_{all} := MSTC48B3 = 3.975 \cdot kip$ Allowable tension load (Simpson **MSTC48B3**)

$$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right) \qquad ratio := \frac{T}{T_{all}} = 0.2 \qquad check_{HD} = "OK"$$

Use MSTC48B3 w/ (14) 10d to face of beam, (4) 10d to bottom of beam, & (38) into (2) 2x min post stitch nailed

SW 1b IN - PLANE SHEAR



$h_t := 8 \cdot ft$ Wall height

$L_s := 4.75ft + 2.5ft$ Total shear wall length

$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Lrf\_1} = 1.29 \cdot kip$ Reaction at wall line

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

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 4.75ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 1.68$ $check_{ratio} := if\left(\frac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $check_{ratio} = "OK"$

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

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $V_{rf} = 0.51 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 4.1 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ Roof load $P_{rf} = 0.07 \cdot kip$

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

$M_{res} := \left[(P_{rf} + P_w) \cdot \frac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $M_{res} = 0.64 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 107 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right) \qquad check_{wv} = "OK"$$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.053 \cdot kip$ Shear load to each anchor

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.24 \qquad Check_a = "OK"$$

Use 16d Nail at 6"o.c. Staggered

Holdown

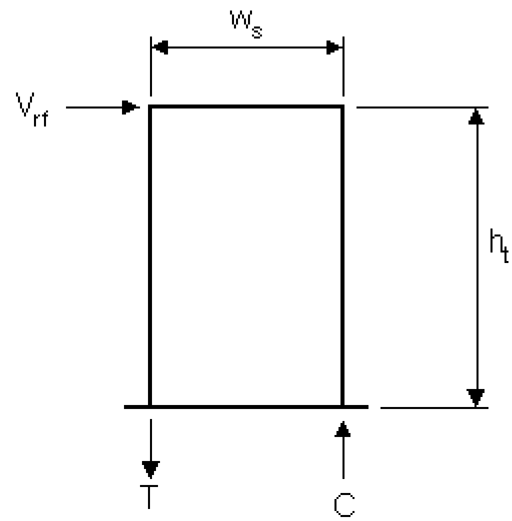
$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.72 \cdot kip \qquad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D") \qquad check_T = "HD REQ'D"$$

$T_{all} := MST37 = 2.71 \cdot kip$ Allowable tension load (Simpson **MST37**)

$$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right) \qquad ratio := \frac{T}{T_{all}} = 0.26 \qquad check_{HD} = "OK"$$

Use Simpson MST37 w/ (22) 16d Nails into (2) 2x min Post Stitch Nailed

SW2 IN - PLANE SHEAR



$h_t := 8 \cdot ft$ Wall height

$L_s := 15.58ft$ Total shear wall length

$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Lrf\_2} = 2.82 \cdot kip$ Reaction at wall line

$w_{rf} := 2ft$ Tributary width of roof on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 15.58ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 0.51$ $\quad check_{ratio} := if\left(\frac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $\quad check_{ratio} = "OK"$

$(WSP) := if\left(\frac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \frac{h_t}{w_s}\right)$ Aspect ratio factor $\quad (WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $\quad V_{rf} = 1.69 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $\quad M_{ot} = 13.6 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ Roof load $\quad P_{rf} = 0.47 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $\quad P_w = 1.25 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \frac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad M_{res} = 8.01 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 109 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf$$

$$check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$$

$check_{wv} = "OK"$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.054 \cdot kip$ Shear load to each anchor

$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right)$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.24$

$Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.36 \cdot kip$$

$check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D")$ $check_T = "HD REQ'D"$

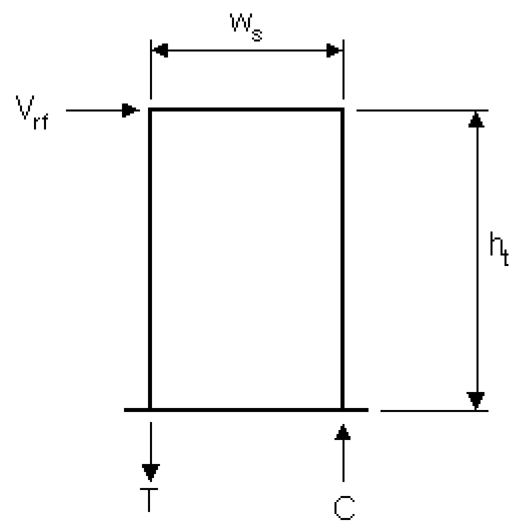
$T_{all} := MST37 = 2.71 \cdot kip$ Allowable tension load (Simpson **MST37**)

$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right)$ $ratio := \frac{T}{T_{all}} = 0.13$

$check_{HD} = "OK"$

Use Simpson MST37 w/ (22) 16d Nails into (2) 2x min Post Stitch Nailed

SW3 IN - PLANE SHEAR



$h_t := 8 \cdot ft$ — Wall height

$L_s := 3ft + 4ft$ — Total shear wall length

$DL_{rf} = 15 \cdot psf$ — Dead load of roof

$R := R_{Lrf\_3} = 1.54 \cdot kip$ — Reaction at wall line

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

$p_{ext\_w} = 10 \cdot psf$ — Dead load of exterior walls

$w_s := 3ft$ — Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 2.67$

$check_{ratio} := if\left(\frac{h_t}{w_s} > 3.5, "NG", "OK"\right)$

$check_{ratio} = "OK"$

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

$(WSP) = 0.9$

Overturning Forces

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

$V_{rf} = 0.4 \cdot kip$

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

$M_{ot} = 3.2 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ — Roof load

$P_{rf} = 0.05 \cdot kip$

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

$P_w = 0.24 \cdot kip$

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

$M_{res} = 0.26 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 132 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 398.8 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right) \qquad check_{wv} = "OK"$$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.066 \cdot kip$ Shear load to each anchor

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.29 \qquad Check_a = "OK"$$

Use 16d Nail at 6"o.c. Staggered

Holdown

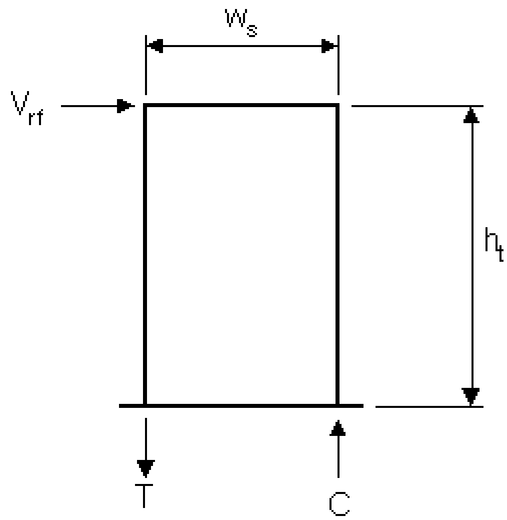
$$T := \frac{M_{ot} - M_{res}}{w_s} = 0.97 \cdot kip \qquad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D") \qquad check_T = "HD REQ'D"$$

$T_{all} := MSTC48B3 = 3.975 \cdot kip$ Allowable tension load (Simpson **MST48B3**)

$$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right) \qquad ratio := \frac{T}{T_{all}} = 0.24 \qquad check_{HD} = "OK"$$

Use MSTC48B3 w/ (14) 10d to face of beam, (4) 10d to bottom of beam, & (38) into (2) 2x min post stitch nailed

SWA IN - PLANE SHEAR



$h_t := 8 \cdot ft$ Wall height

$L_s := 10.5ft + 5.33ft + 24ft$ Total shear wall length

$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Trf\_A} = 1.38 \cdot kip$ Reaction at wall line

$w_{rf} := \dfrac{25ft}{2}$ Tributary width of roof on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 5.33ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 1.5$ $\quad check_{ratio} := \text{if}\left(\dfrac{h_t}{w_s} > 3.5, \text{"NG"}, \text{"OK"}\right)$ $\quad check_{ratio} = \text{"OK"}$

$(WSP) := \text{if}\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ Aspect ratio factor $\quad (WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $\quad V_{rf} = 0.11 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $\quad M_{ot} = 0.9 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ Roof load $\quad P_{rf} = 1 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $\quad P_w = 0.43 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad M_{res} = 2.28 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 21 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf$$

$$check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$$

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

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.01 \cdot kip$ Shear load to each anchor

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.05$ $Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -0.26 \cdot kip \quad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D")$$

$$check_T = "NOT REQ'D"$$

SWB IN - PLANE SHEAR

$h_t := 8 \cdot ft$ Wall height

$L_s := 3.67ft + 4ft + 19.42ft + 4.5ft + 2.5ft$ Total shear wall length

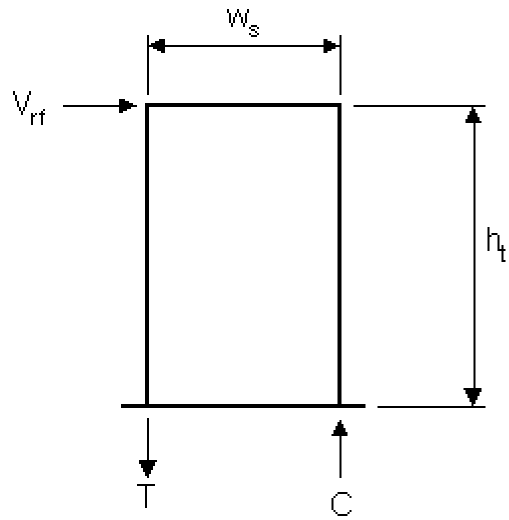
$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Trf\_A} = 1.38 \cdot kip$ Reaction at wall line

$w_{rf} := \frac{25ft}{2}$ Tributary width of roof on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 2.5ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\frac{h_t}{w_s} = 3.2$

$check_{ratio} := if\left(\frac{h_t}{w_s} > 3.5, "NG", "OK"\right)$

$check_{ratio} = "OK"$

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

$(WSP) = 0.9$

Overturning Forces

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

$V_{rf} = 0.06 \cdot kip$

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

$M_{ot} = 0.5 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s)$ Roof load

$P_{rf} = 0.47 \cdot kip$

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

$P_w = 0.2 \cdot kip$

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

$M_{res} = 0.5 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 24 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 369.8 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, \text{"NG"}, \text{"OK"}\right) \qquad check_{wv} = \text{"OK"}$$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing $C_D := 1.6$

$t_{sp} := 1.5in$ Bottom plate thickness $dia_a := 16d$ Fastener Type/Size $sp_a := 6in$ Fastener spacing

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

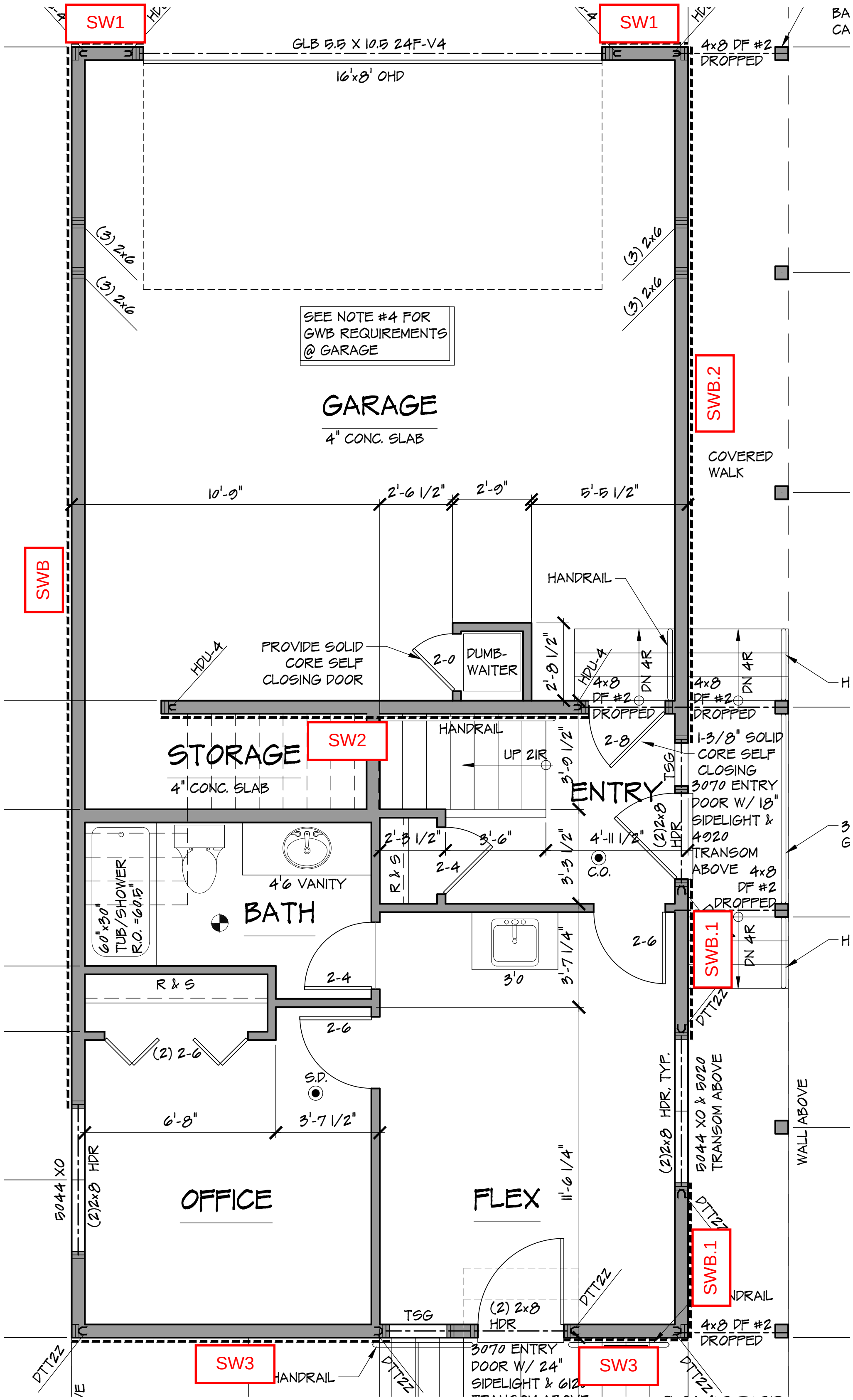
$V_{sp} := w_v \cdot sp_a = 0.012 \cdot kip$ Shear load to each anchor

$$Check_a := if\left(V_{sp} > Z_{ll}, \text{"NG"}, \text{"OK"}\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.05 \qquad Check_a = \text{"OK"}$$

Use 16d Nail at 6"o.c. Staggered

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -5.64 \times 1(\quad check_T := if(T > 150lbf, \text{"HD REQ'D"}, \text{"NOT REQ'D"}) \qquad check_T = \text{"NOT REQ'D"}$$



NKH ENGINEERING

PROJECT: Gass Apartment

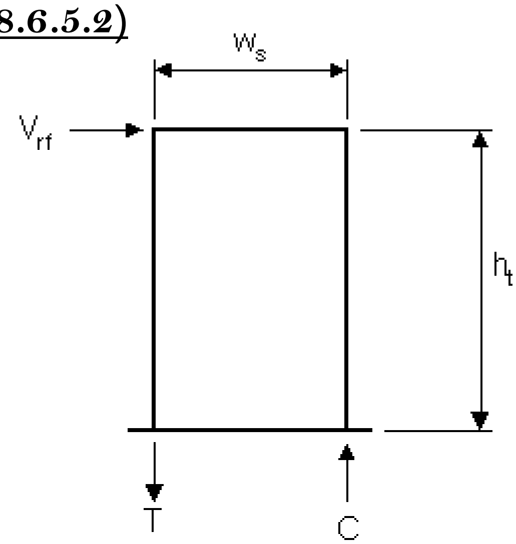
DESCRIPTION: Main Floor Shearwall Keyplan

BY: AKR DATE: 1/25/2021 JOB #: 20-172

S25

Shear Wall Check - Main to Upper Floor (ref. ANSI/AF&PA SDPWS-2015)

SW1 IN - PLANE SHEAR (PFH per IBC 2308.6.5.2)



$h_t := 13 \cdot ft$	Wall height
$h_{t\_pfh} := 8.5ft$	Portal Frame Height
$L_{pfh} := 16ft$	Clear Span of Header
$L_s := 30in + 30in$	Total shear wall length
$DL_{rf} = 15 \cdot psf$	Dead load of roof
$R := R_{Lup\_1} = 3.9 \cdot kip$	Reaction at wall line
$w_{rf} := \frac{2ft + 5.5ft}{2} + 3ft$	Tributary width of framing on wall
$p_{ext\_w} = 10 \cdot psf$	Dead load of exterior walls
$w_s := 30in$	Shear wall length

Dimensional Checks (PFH)

$check_{height} := if(h_{t\_pfh} \le 10ft, "OK", "NG")$ $\quad check_{height} = "OK"$

$check_{width} := if(w_s \ge 16in, "OK", "NG")$ $\quad check_{width} = "OK"$

$check_{span} := if(6ft \le L_{pfh} \le 18ft, "OK", "NG")$ $\quad check_{span} = "OK"$

Overturning Forces

$V_{rf} := \left(R \cdot \frac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD Wind) $\quad V_{rf} = 1.16 \cdot kip$

$M_{ot} := V_{rf} \cdot h_{t\_pfh}$ Overturning moment (ASD) $\quad M_{ot} = 9.9 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := DL_{rf} \cdot w_{rf} \cdot (w_s + 4ft)$ Roof load $\quad P_{rf} = 0.66 \cdot kip$

$P_w := p_{ext\_w} \cdot (2h_t) \cdot (w_s + 4ft)$ Wall load $\quad P_w = 1.69 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \frac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad M_{res} = 1.76 \cdot kip \cdot ft$

Sill Plate Anchorage

$C_D := 1.6$ Load Duration Factor

$w_v := \frac{V_{rf}}{w_s} = 464 \cdot plf$ Shear Load

$t_{sp} := 1.5in$ Sill plate thickness

$dia_a := 0.625in$ Anchor Diameter

$sp_a := 24in$ Anchor spacing

$Z_{ll} := v_{A.625\_2x} \cdot C_D = 1.49 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.927 \cdot kip$ Shear load to each anchor

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $\quad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.62$ $\quad Check_a = "OK"$

Use 5/8" Dia. Anchor at the center of wall in 2x sill plate (7" min. embed)

Holdown

$T := \frac{M_{ot} - M_{res}}{w_s} = 3.24 \cdot kip$ $\quad check_T := if(T > 110lbf, "HD REQ'D", "NOT REQ'D")$ $\quad check_T = "HD REQ'D"$

$T_{all} := HDU4 = 4.565 \cdot kip$ Allowable tension load (Simpson **HDU4**)

$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right)$ $\quad ratio := \frac{T}{T_{all}} = 0.71$ $\quad check_{HD} = "OK"$

Use Simpson HDU4 each end of portal frame leg w/ 5/8" Dia. Anchor, 12" embed (Ref. Anchor Output)

Footing

$L_{ftg} := w_s + 8ft = 10.5\ ft$ Length of footing

$t_{slab} := 4in$ Slab thickness

$W_{ftg} := 1.33ft$ Width of footing

$trib_{slab} := 8ft$ Slab tributary

$t_{ftg} := 10in$ Thickness of footing

$t_{stem} := 6in$ Stem wall thick

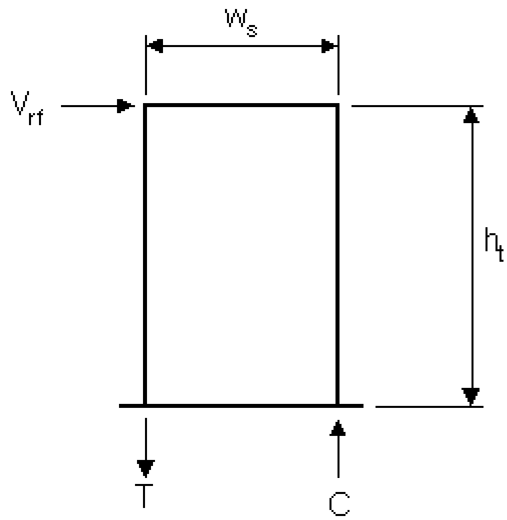
$ht_{stem} := 18in$ Stem wall height

$wt_{resist} := (W_{ftg} \cdot t_{ftg} + t_{slab} \cdot trib_{slab} + t_{stem} \cdot ht_{stem}) \cdot \frac{L_{ftg}}{2} \cdot 150pcf = 3.56 \cdot kip$ Weight resisting uplift

$check_{ftg} := if(wt_{resist} > T, "OK", "NG")$ $\quad ratio := \frac{T}{wt_{resist}} = 0.91$ $\quad check_{ftg} = "OK"$

Use 1'-4"" Wide x 10" Deep footing w/ (2) #4 Longitudinal & #4 @ 10" o.c. Transverse

SW2 IN - PLANE SHEAR



$h_t := 12 \cdot ft$	Wall height
$L_s := 14ft$	Total shear wall length
$DL_{rf} = 15 \cdot psf$	Dead load of roof
$R := R_{Lup\_2} = 7.84 \cdot kip$	Reaction at wall line
$w_{rf} := \dfrac{15.5ft + 4ft}{2}$	Tributary width of framing on wall
$p_{ext\_w} = 10 \cdot psf$	Dead load of exterior walls
$w_s := 14ft$	Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 0.86$ $\quad$ $check_{ratio} := if\left(\dfrac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $\quad$ $check_{ratio} = "OK"$

$(WSP) := if\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ Aspect ratio factor $\quad$ $(WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $\quad$ $V_{rf} = 4.7 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $\quad$ $M_{ot} = 56.4 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := (DL_{rf}) \cdot w_{rf} \cdot (w_s)$ Roof load $\quad$ $P_{rf} = 2.05 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $\quad$ $P_w = 1.68 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad$ $M_{res} = 15.66 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 336 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf$$

$$check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$$

$check_{wv} = "OK"$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C.
Interior Supports (ref. table 4.3A)

Bottom Plate Nailing

$C_D := 1.6$

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

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.168 \cdot kip$ Shear load to each nail

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.74$ $Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Sill Plate Anchorage

$C_D := 1.6$

$t_{sp} := 1.5in$ Sill plate thickness $dia_a := 0.5in$ Anchor Diameter $sp_a := 36in$ Anchor spacing

$Z_{ll} := v_{A.5\_2x} \cdot C_D = 1.04 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 1.008 \cdot kip$ Shear load to each anchor

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.97$ $Check_a = "OK"$

Use 1/2" Dia. Anchor at 36"o.c. (7" min. embed)

Holdown

$T := \dfrac{M_{ot} - M_{res}}{w_s} = 2.91 \cdot kip$ $\quad check_T := if(T > 150lbf, \text{"HD REQ'D"}, \text{"NOT REQ'D"})$ $\quad check_T = \text{"HD REQ'D"}$

$T_{all} := HDU4 = 4.565 \cdot kip$ Allowable tension load (Simpson **HDU4**)

$check_{HD} := if\left(\dfrac{T}{T_{all}} > 1.0, \text{"NG"}, \text{"OK"}\right)$ $\quad ratio := \dfrac{T}{T_{all}} = 0.64$ $\quad check_{HD} = \text{"OK"}$

Anchor

$T_{LRFD} := \dfrac{\dfrac{M_{ot}}{0.6} - M_{res} \cdot \dfrac{0.9}{0.6}}{w_s}$ Tension in anchor bolt (LRFD) $\quad T_{LRFD} = 5.04 \cdot kip$

Use Simpson HDU4 w/ 5/8" Dia. Anchor, 12" min. embed (Ref. Anchor Output)

Footing Uplift

$L_{ftg} := w_s + 6ft = 20\,ft$	Length of footing	$t_{slab} := 4in$	Slab thickness
$W_{ftg} := 1.33ft$	Width of footing	$trib_{slab} := 6ft$	Slab tributary
$t_{ftg} := 10in$	Thickness of footing	$t_{stem} := 8in$	Stem wall thick
$trib_{flr} := 0ft$	Floor/deck tributary	$ht_{stem} := 18in$	Stem wall height

$wt_{resist} := \left[\left(W_{ftg} \cdot t_{ftg} + t_{slab} \cdot trib_{slab} + t_{stem} \cdot ht_{stem}\right) \cdot 150pcf + trib_{flr} \cdot DL_{flr}\right] \cdot \dfrac{L_{ftg}}{2} = 6.16 \cdot kip$ Weight resisting uplift

$check_{ftg} := if\left(wt_{resist} > T, \text{"OK"}, \text{"NG"}\right)$ $\quad ratio := \dfrac{T}{wt_{resist}} = 0.47$ $\quad check_{ftg} = \text{"OK"}$

Use 1'-4"W x 6"D footing w/ (2) #4 Long., #4 @ 10" o.c. Trans

SW3 IN - PLANE SHEAR

$h_t := 12 \cdot ft$ Wall height

$L_s := 10.75ft + 4.25ft$ Total shear wall length

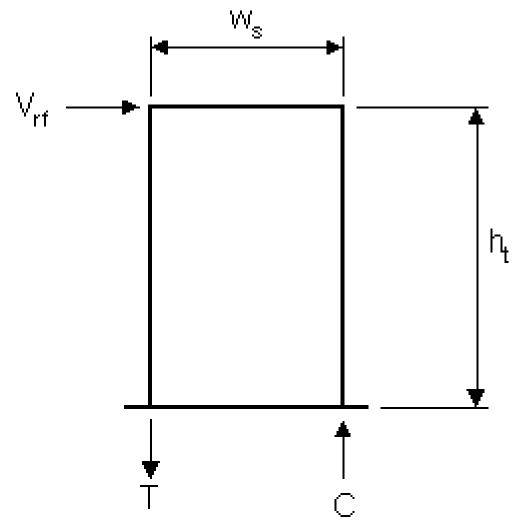
$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Lup\_3} = 4.64 \cdot kip$ Reaction at wall line

$w_{rf} := \dfrac{17.5ft + 2ft}{2} + 6ft$ Tributary width of framing on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 4.25ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 2.82$ $\quad check_{ratio} := if\left(\dfrac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $\quad check_{ratio} = "OK"$

$(WSP) := if\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ Aspect ratio factor $\quad (WSP) = 0.9$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $\quad V_{rf} = 0.79 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $\quad M_{ot} = 9.5 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := (DL_{rf}) \cdot w_{rf} \cdot (w_s)$ Roof load $\quad P_{rf} = 1 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $\quad P_w = 0.51 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad M_{res} = 1.93 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$w_v := \dfrac{V_{rf}}{w_s} = 186 \cdot plf$

$w_{all} := \dfrac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 390.2 \cdot plf$ $check_{wv} := if\left(\dfrac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$ $check_{wv} = "OK"$

<u>Single Sided</u> 15/32" sheathing w/ 10d @ **<u>6" O.C.</u>** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing

$C_D := 1.6$

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

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.093 \cdot kip$ Shear load to each nail

$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right)$ $ratio_a := \dfrac{V_{sp}}{Z_{ll}} = 0.41$ $Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Sill Plate Anchorage

$C_D := 1.6$

$t_{sp} := 1.5in$ Sill plate thickness $dia_a := 0.5in$ Anchor Diameter $sp_a := 36in$ Anchor spacing

$Z_{ll} := v_{A.5\_2x} \cdot C_D = 1.04 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.557 \cdot kip$ Shear load to each anchor

$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right)$ $ratio_a := \dfrac{V_{sp}}{Z_{ll}} = 0.54$ $Check_a = "OK"$

Use 1/2" Dia. Anchor at 36"o.c. (7" min. embed)

Holdown

$T := \dfrac{M_{ot} - M_{res}}{w_s} = 1.77 \cdot kip$ $check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D")$ $check_T = "HD REQ'D"$

$T_{all} := DTT2Z = 2.145 \cdot kip$ Allowable tension load (Simpson **DTT2Z**)

$check_{HD} := if\left(\dfrac{T}{T_{all}} > 1.0, "NG", "OK"\right)$ $ratio := \dfrac{T}{T_{all}} = 0.83$ $check_{HD} = "OK"$

Anchor

$T_{LRFD} := \dfrac{\dfrac{M_{ot}}{0.6} - M_{res} \cdot \dfrac{0.9}{0.6}}{w_s}$ Tension in anchor bolt (LRFD) $T_{LRFD} = 3.03 \cdot kip$

Use Simpson DTT2Z w/ 1/2" Dia. Anchor, 10" min. embed (Ref. Anchor Output)

Footing Uplift

$L_{ftg} := w_s + 6ft = 10.25\,ft$ Length of footing $t_{slab} := 4in$ Slab thickness

$W_{ftg} := 1.33ft$ Width of footing $trib_{slab} := 6ft$ Slab tributary

$t_{ftg} := 10in$ Thickness of footing $t_{stem} := 6in$ Stem wall thick

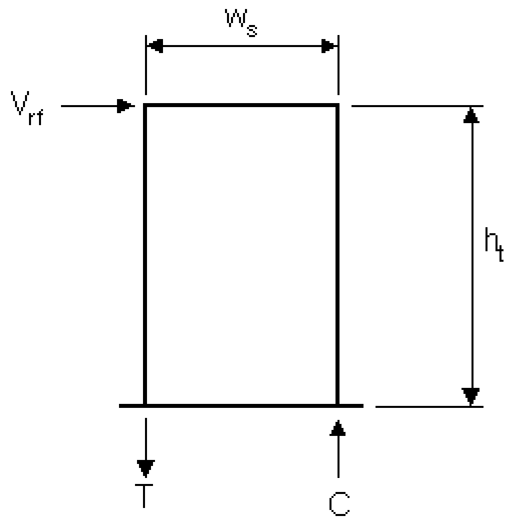
$trib_{flr} := 0ft$ Floor/deck tributary $ht_{stem} := 18in$ Stem wall height

$wt_{resist} := \left[\left(W_{ftg} \cdot t_{ftg} + t_{slab} \cdot trib_{slab} + t_{stem} \cdot ht_{stem}\right) \cdot 150pcf + trib_{flr} \cdot DL_{flr}\right] \cdot \dfrac{L_{ftg}}{2} = 2.97 \cdot kip$ Weight resisting uplift

$check_{ftg} := if\left(wt_{resist} > T, "OK", "NG"\right)$ $ratio := \dfrac{T}{wt_{resist}} = 0.6$ $check_{ftg} = "OK"$

Use 1'-4"W x 6"D footing w/ (2) #4 Long., #4 @ 10" o.c. Trans

SWA IN - PLANE SHEAR



$h_t := 12 \cdot ft$ Wall height

$L_s := 37ft$ Total shear wall length

$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Tup\_A} = 3.77 \cdot kip$ Reaction at wall line

$w_{rf} := \dfrac{25ft + 2ft}{2}$ Tributary width of framing on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 37ft$ Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 0.32$ $check_{ratio} := if\left(\dfrac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $check_{ratio} = "OK"$

$(WSP) := if\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ Aspect ratio factor $(WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $V_{rf} = 2.26 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $M_{ot} = 27.1 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := \left(DL_{rf}\right) \cdot w_{rf} \cdot \left(w_s\right)$ Roof load $P_{rf} = 7.49 \cdot kip$

$P_w := p_{ext\_w} \cdot \left(h_t\right) \cdot \left(w_s\right)$ Wall load $P_w = 4.44 \cdot kip$

$M_{res} := \left[\left(P_{rf} + P_w\right) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $M_{res} = 132.45 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 61 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right) \qquad check_{wv} = "OK"$$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing

$C_D := 1.6$

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

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.031 \cdot kip$ Shear load to each nail

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.14 \qquad Check_a = "OK"$$

Use 16d Nail at 6"o.c. Staggered

Sill Plate Anchorage

$C_D := 1.6$

$t_{sp} := 1.5in$ Sill plate thickness $dia_a := 0.5in$ Anchor Diameter $sp_a := 36in$ Anchor spacing

$Z_{ll} := v_{A.5\_2x} \cdot C_D = 1.04 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.183 \cdot kip$ Shear load to each anchor

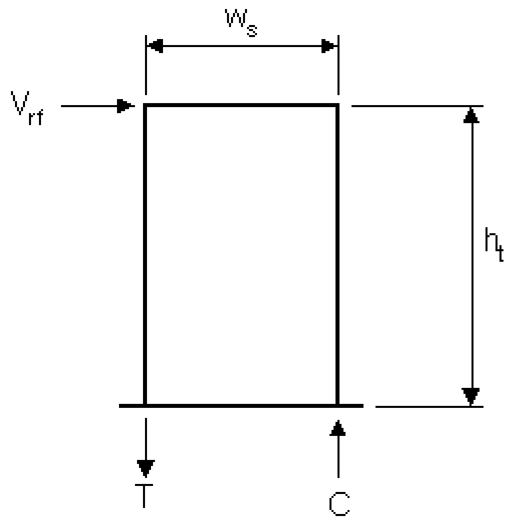
$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.18 \qquad Check_a = "OK"$$

Use 1/2" Dia. Anchor at 36"o.c. (7" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -2.85 \cdot kip \quad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D") \quad check_T = "NOT REQ'D"$$

SWB.1 IN - PLANE SHEAR



$h_t := 12 \cdot ft$ — Wall height

$L_s := 5.33ft + 5.67ft + 24ft$ — Total shear wall length

$DL_{rf} = 15 \cdot psf$ — Dead load of roof

$R := R_{Tup\_B} = 4.15 \cdot kip$ — Reaction at wall line

$w_{rf} := \dfrac{25ft + 4ft}{2}$ — Tributary width of framing on wall

$p_{ext\_w} = 10 \cdot psf$ — Dead load of exterior walls

$w_s := 5.33ft$ — Shear wall length

Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 2.25$

$check_{ratio} := if\left(\dfrac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ — $check_{ratio} = "OK"$

$(WSP) := if\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ — Aspect ratio factor — $(WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ — Shear load at top of wall (ASD) — $V_{rf} = 0.38 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ — Overturning moment (ASD) — $M_{ot} = 4.6 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := \left(DL_{rf}\right) \cdot w_{rf} \cdot \left(w_s\right)$ — Roof load — $P_{rf} = 1.16 \cdot kip$

$P_w := p_{ext\_w} \cdot \left(h_t\right) \cdot \left(w_s\right)$ — Wall load — $P_w = 0.64 \cdot kip$

$M_{res} := \left[\left(P_{rf} + P_w\right) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ — Resisting moment (ASD) — $M_{res} = 2.88 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$w_v := \frac{V_{rf}}{w_s} = 71 \cdot plf$

$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 421.3 \cdot plf$ $check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$ $check_{wv} = "OK"$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing

$C_D := 1.6$

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

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.036 \cdot kip$ Shear load to each nail

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.16$ $Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Sill Plate Anchorage

$C_D := 1.6$

$t_{sp} := 1.5in$ Sill plate thickness $dia_a := 0.5in$ Anchor Diameter $sp_a := 36in$ Anchor spacing

$Z_{ll} := v_{A.5\_2x} \cdot C_D = 1.04 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.214 \cdot kip$ Shear load to each anchor

$Check_a := if(V_{sp} > Z_{ll}, "NG", "OK")$ $ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.21$ $Check_a = "OK"$

Use 1/2" Dia. Anchor at 36"o.c. (7" min. embed)

Holdown

$T := \frac{M_{ot} - M_{res}}{w_s} = 0.31 \cdot kip$ $check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D")$ $check_T = "HD REQ'D"$

$T_{all} := DTT2Z = 2.145 \cdot kip$ Allowable tension load (Simpson **DTT2Z**)

$check_{HD} := if\left(\frac{T}{T_{all}} > 1.0, "NG", "OK"\right)$ $ratio := \frac{T}{T_{all}} = 0.15$ $check_{HD} = "OK"$

Anchor

$T_{LRFD} := \frac{\frac{M_{ot}}{0.6} - M_{res} \cdot \frac{0.9}{0.6}}{w_s}$ Tension in anchor bolt (LRFD) $T_{LRFD} = 0.61 \cdot kip$

Use Simpson DTT2Z w/ 1/2" Dia. Anchor, 10" min. embed (Ref. Anchor Output)

Footing Uplift

$L_{ftg} := w_s + 6ft = 11.33\ ft$	Length of footing	$t_{slab} := 0in$	Slab thickness
$W_{ftg} := 1.33ft$	Width of footing	$trib_{slab} := 6ft$	Slab tributary
$t_{ftg} := 10in$	Thickness of footing	$t_{stem} := 6in$	Stem wall thick
$trib_{flr} := 2ft$	Floor/deck tributary	$ht_{stem} := 18in$	Stem wall height

$wt_{resist} := \left[\left(W_{ftg} \cdot t_{ftg} + t_{slab} \cdot trib_{slab} + t_{stem} \cdot ht_{stem}\right) \cdot 150pcf + trib_{flr} \cdot DL_{flr}\right] \cdot \frac{L_{ftg}}{2} = 1.75 \cdot kip$ Weight resisting uplift

$check_{ftg} := if\left(wt_{resist} > T, "OK", "NG"\right)$ $ratio := \frac{T}{wt_{resist}} = 0.18$ $check_{ftg} = "OK"$

Use 1'-4"W x 6"D footing w/ (2) #4 Long., #4 @ 10" o.c. Trans

SWB.2 IN - PLANE SHEAR

$h_t := 12 \cdot ft$ Wall height

$L_s := 5.33ft + 5.67ft + 24ft$ Total shear wall length

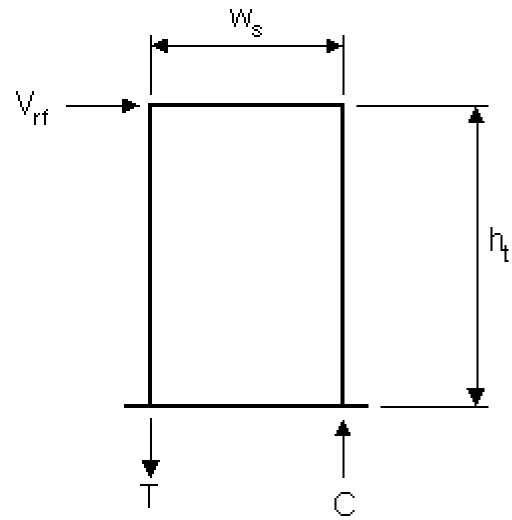
$DL_{rf} = 15 \cdot psf$ Dead load of roof

$R := R_{Tup\_B} = 4.15 \cdot kip$ Reaction at wall line

$w_{rf} := \dfrac{25ft + 4ft}{2}$ Tributary width of framing on wall

$p_{ext\_w} = 10 \cdot psf$ Dead load of exterior walls

$w_s := 24ft$ Shear wall length



Aspect Ratio (Blocked Shear Wall)

$\dfrac{h_t}{w_s} = 0.5$ $\quad check_{ratio} := if\left(\dfrac{h_t}{w_s} > 3.5, "NG", "OK"\right)$ $\quad check_{ratio} = "OK"$

$(WSP) := if\left(\dfrac{h_t}{w_s} < 2.0, 1.0, 1.25 - 0.125 \cdot \dfrac{h_t}{w_s}\right)$ Aspect ratio factor $\quad (WSP) = 1.0$

Overturning Forces

$V_{rf} := \left(R \cdot \dfrac{w_s}{L_s}\right) 0.6$ Shear load at top of wall (ASD) $\quad V_{rf} = 1.71 \cdot kip$

$M_{ot} := V_{rf} \cdot h_t$ Overturning moment (ASD) $\quad M_{ot} = 20.5 \cdot kip \cdot ft$

Resisting Forces

$P_{rf} := (DL_{rf}) \cdot w_{rf} \cdot (w_s)$ Roof load $\quad P_{rf} = 5.22 \cdot kip$

$P_w := p_{ext\_w} \cdot (h_t) \cdot (w_s)$ Wall load $\quad P_w = 2.88 \cdot kip$

$M_{res} := \left[(P_{rf} + P_w) \cdot \dfrac{w_s}{2}\right] \cdot 0.6$ Resisting moment (ASD) $\quad M_{res} = 58.32 \cdot kip \cdot ft$

Plywood Shear (ref. ANSI/AF&PA SDPWS)

$\Omega_s := 2.0$ (ref. section 4.3.3) $n := 1$ sides

$$w_v := \frac{V_{rf}}{w_s} = 71 \cdot plf$$

$$w_{all} := \frac{(WSP) \cdot v_{w6} \cdot n}{\Omega_s} = 435 \cdot plf \qquad check_{wv} := if\left(\frac{w_v}{w_{all}} > 1.0, "NG", "OK"\right)$$

$check_{wv} = "OK"$

Single Sided 15/32" sheathing w/ 10d @ **6" O.C.** Panel Edges @ 12" O.C. Interior Supports (ref. table 4.3A)

Bottom Plate Nailing

$C_D := 1.6$

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

$Z_{ll} := v_n \cdot C_D = 0.23 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.036 \cdot kip$ Shear load to each nail

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.16$$

$Check_a = "OK"$

Use 16d Nail at 6"o.c. Staggered

Sill Plate Anchorage

$C_D := 1.6$

$t_{sp} := 1.5in$ Sill plate thickness $dia_a := 0.5in$ Anchor Diameter $sp_a := 36in$ Anchor spacing

$Z_{ll} := v_{A.5\_2x} \cdot C_D = 1.04 \cdot kip$ Allowable load parallel to grain (ref. NDS table 12)

$V_{sp} := w_v \cdot sp_a = 0.214 \cdot kip$ Shear load to each anchor

$$Check_a := if\left(V_{sp} > Z_{ll}, "NG", "OK"\right) \qquad ratio_a := \frac{V_{sp}}{Z_{ll}} = 0.21$$

$Check_a = "OK"$

Use 1/2" Dia. Anchor at 36"o.c. (7" min. embed)

Holdown

$$T := \frac{M_{ot} - M_{res}}{w_s} = -1.58 \cdot kip \quad check_T := if(T > 150lbf, "HD REQ'D", "NOT REQ'D")$$

$check_T = "NOT REQ'D"$

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

SIMPSON Strong-Tie®

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.

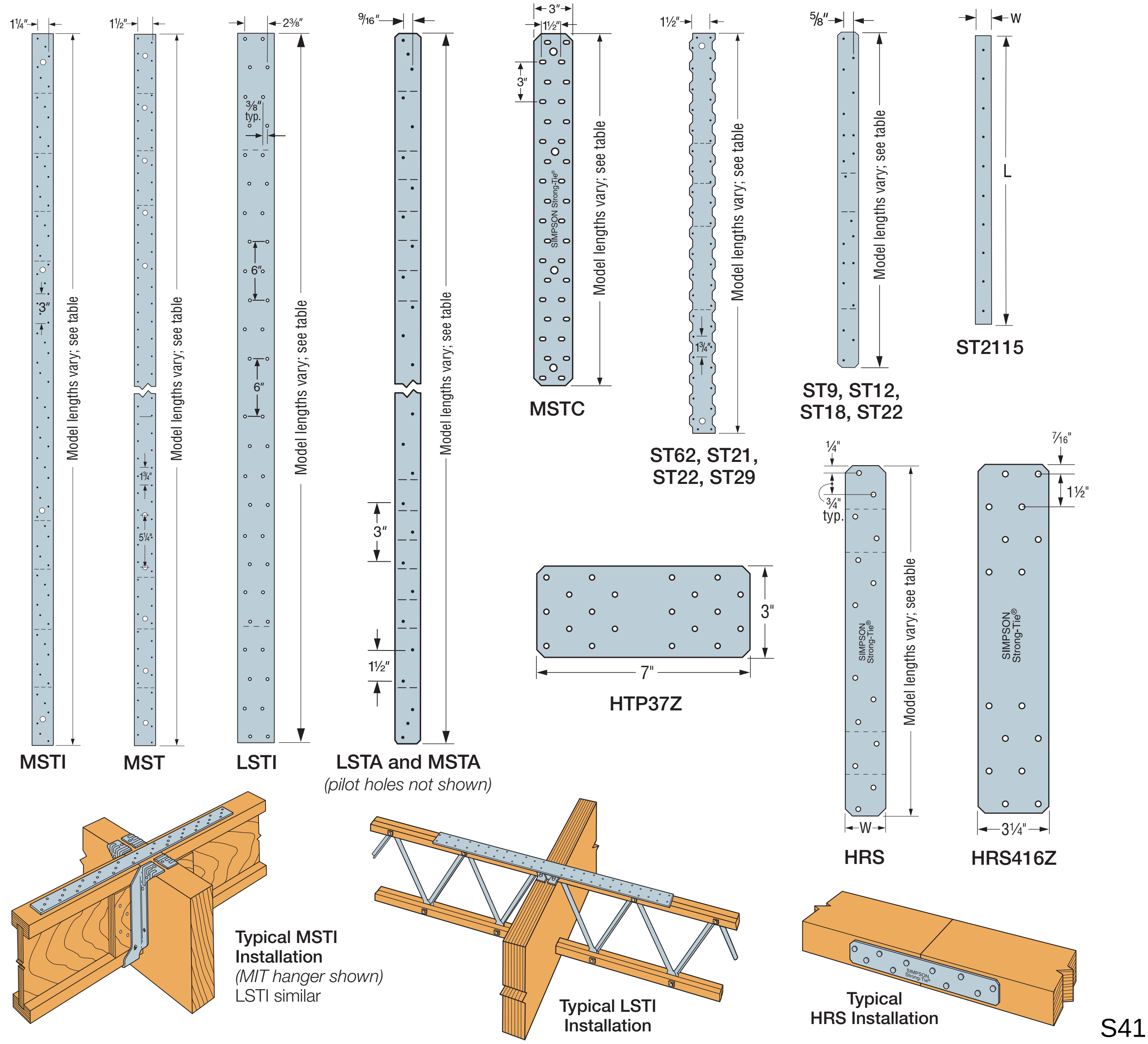
Installation: Use all specified fasteners; see General Notes

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.)



SIMPSON Strong-Tie®

Strap Ties (cont.)

Codes: See p. 12 for Code Reference Key Chart

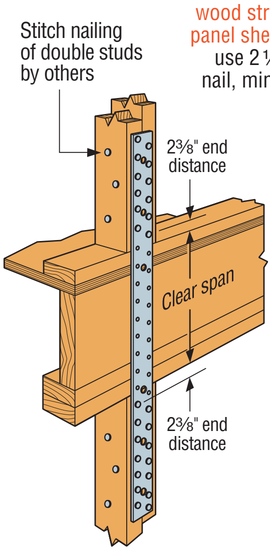
These products are available with additional corrosion protection. For more information, see p. 15.

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

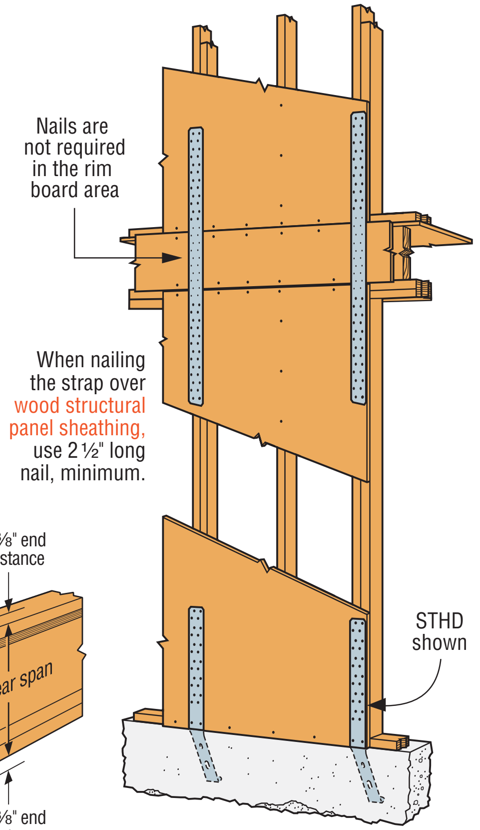
Floor to Floor Span Table

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	Allowable Tension Loads (DF/SP)	Allowable Tension Loads (SPF/HF)
			(160)	(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.)		Fasteners (Total)			Allowable Tension Loads (DF/SP)		Allowable Tension Loads (SPF/HF)		Code Ref.
		W	L	Nails (in.)	Bolts		Nails	Bolts	Nails	Bolts	
					Qty.	Dia.	(160)	(160)	(160)	(160)	
MST27	12	2 1/16	27	(30) 0.162 x 2½	4	½	3,700	2,165	3,210	2,000	IBC, FL, LA
MST37		2 1/16	37½	(42) 0.162 x 2½	6	½	5,070	3,030	4,495	2,800	
MST48		2 1/16	48	(50) 0.162 x 2½	8	½	5,310	3,675	5,190	3,395	
MST60	10	2 1/16	60	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150	
MST72		2 1/16	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.

MSTC48B3/MSTC66B3Z

Pre-Bent Straps

The MSTC48B3 and MSTC66B3Z are pre-bent straps designed to transfer tension load from an upper-story shearwall to a beam on the story below.

Material: 14 gauge

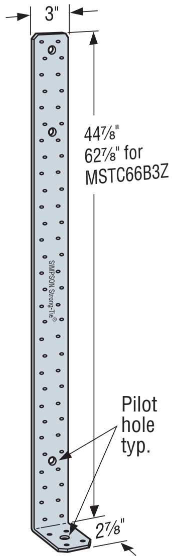
Finish: Galvanized; contact Simpson Strong-Tie

Codes: See p. 12 for Code Reference Key Chart

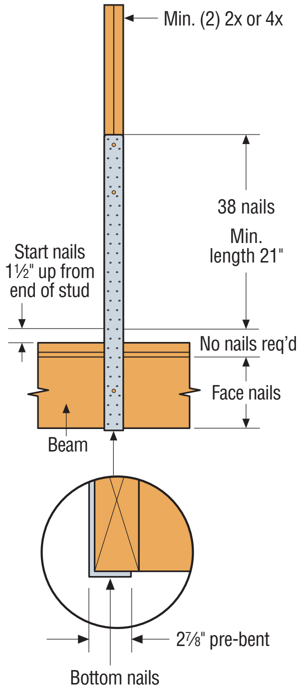
These products are available with additional corrosion protection. For more information, see p. 15.

Model No.	Min. Wood Beam Dimension (in.)		Fasteners (in.)			Allowable Tension Loads		Code Ref.
			Beam		Studs/ Post	DF/SP	SPF/HF	
	Width (min.)	Depth (min.)	Face	Bottom		(160)	(160)	
MSTC48B3	3	9¼	(12) 0.148 x 3	(4) 0.148 x 3	(38) 0.148 x 3	3,975	3,900	IBC, FL, LA
MSTC66B3Z	3½	11¼	(14) 0.148 x 3			4,490	4,490	

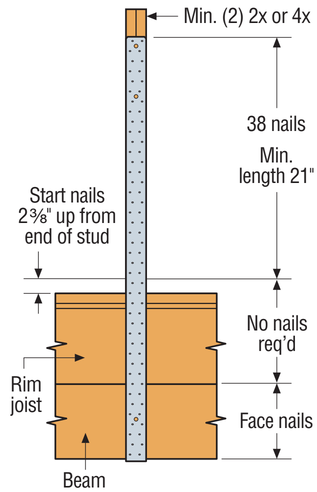
1. Using fewer than 38 nails in the studs/post will reduce the allowable load of the connection. To calculate a reduced allowable load, use 199 lb. per nail for DF/SP or 172 lb. per nail for HF/SPF. Minimum length of extent of reduced nails may not be less than 21" as is shown in graphic.
2. Nails in studs/post shall be installed symmetrically. Nails may be installed over the entire length of the strap in the studs/post.
3. The minimum 3"-wide beam may be made up of two 2x members.
4. MSTC48B3 and MSTC66B3Z installed over wood structural panel sheathing up to ½" thick achieve 0.85 of table loads.
5. PSL beam may be used in lieu of a standard-dimension lumber beam with no load reductions.
6. Multiply allowable loads by 1.85 to attain an allowable load for installations where two straps have been installed with a 1½" clear space between straps.
7. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers known as the narrow face. Values in the tables reflect installation into the wide face. See technical bulletin T-C-SCLCLM at **strongtie.com** for load reductions resulting from narrow-face installations.
8. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.



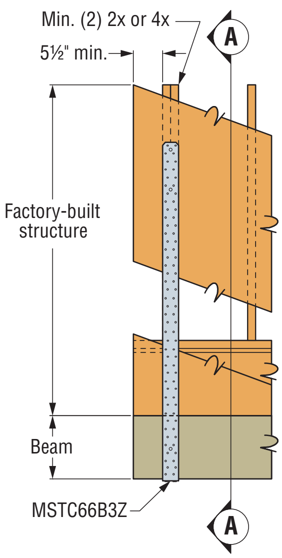
MSTC48B3



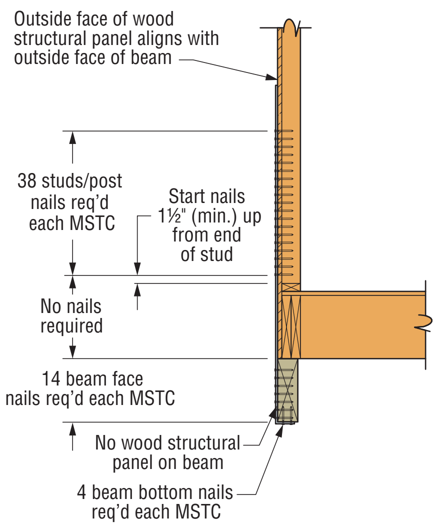
MSTC48B3 Installation with No Rim Board



MSTC66B3Z Installation with Rim Board



(2) MSTC66B3Z Installation



Section A-A

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

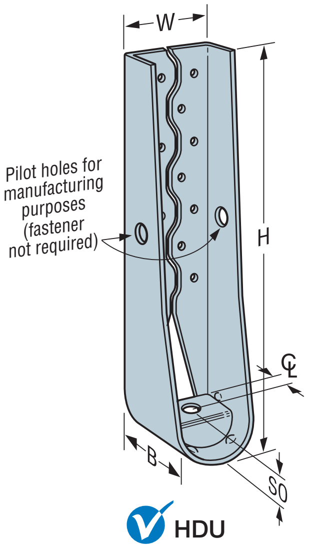
Material: See table

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

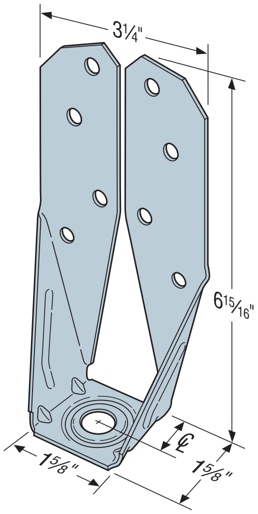
Installation:

- 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.)

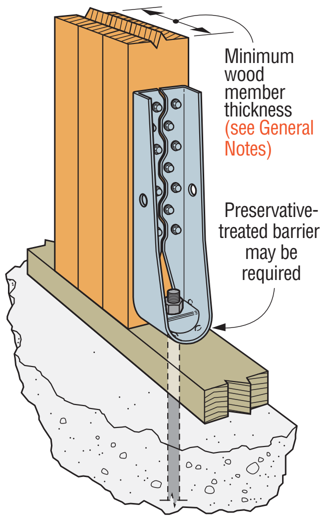
Codes: See p. 12 for Code Reference Key Chart



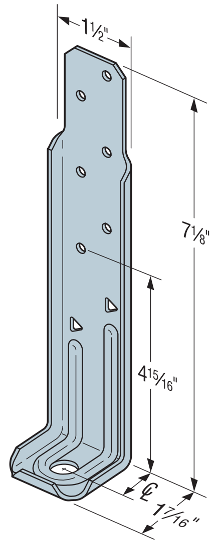
HDU



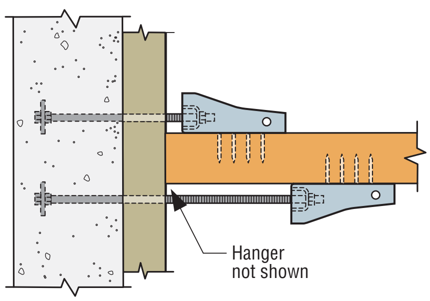
DTT2Z
U.S. Patent 8,555,580



Vertical HDU Installation



DTT1Z
U.S. Patent Pending



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

Holdowns (cont.)

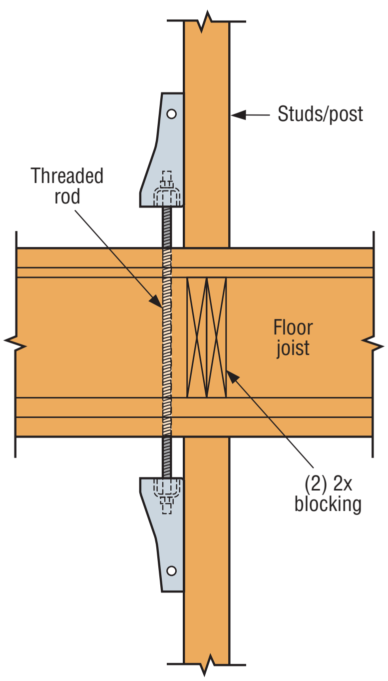
These products are available with additional corrosion protection. For more information, see p. 15.

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.) W	H	B	CL	SO	Fasteners (in.) Anchor Bolt Dia. (in.)	Wood Fasteners	Minimum Wood Member Size (in.)	Allowable Tension Loads (160) DF/SP	SPF/HF	Deflection at Allowable Load (in.)	Code Ref.
	DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	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	6 15/16	1⅝	13/16	3/16	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	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
	HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
	HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	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	25 11/16	3½	1 9/16	1 9/16	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

SIMPSON Strong-Tie®	Anchor Designer™ Software Version 2.9.7376.0

Company:	NKH Engineering	Date:	1/25/2021
Engineer:	A.Rishel	Page:	1/5
Project:	Gass Apartment		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: DTT2Z Anchor
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 10.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.88
C_{min} (inch): 3.00
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 3500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

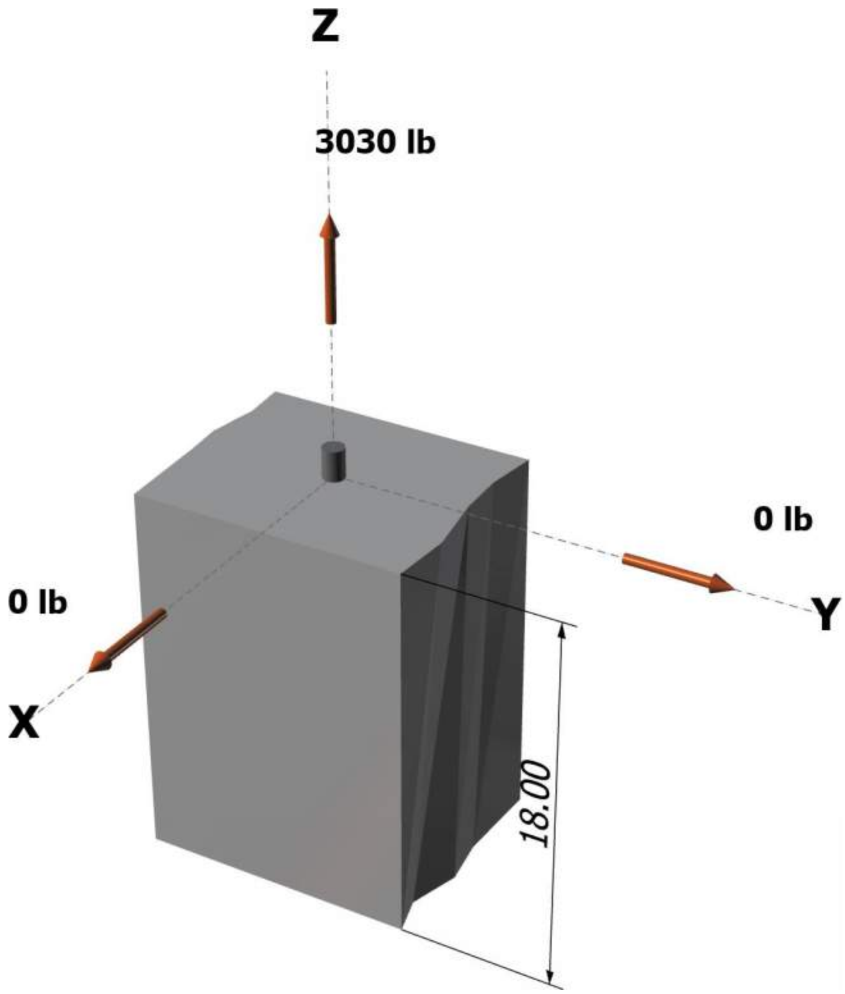
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB4 (1/2"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

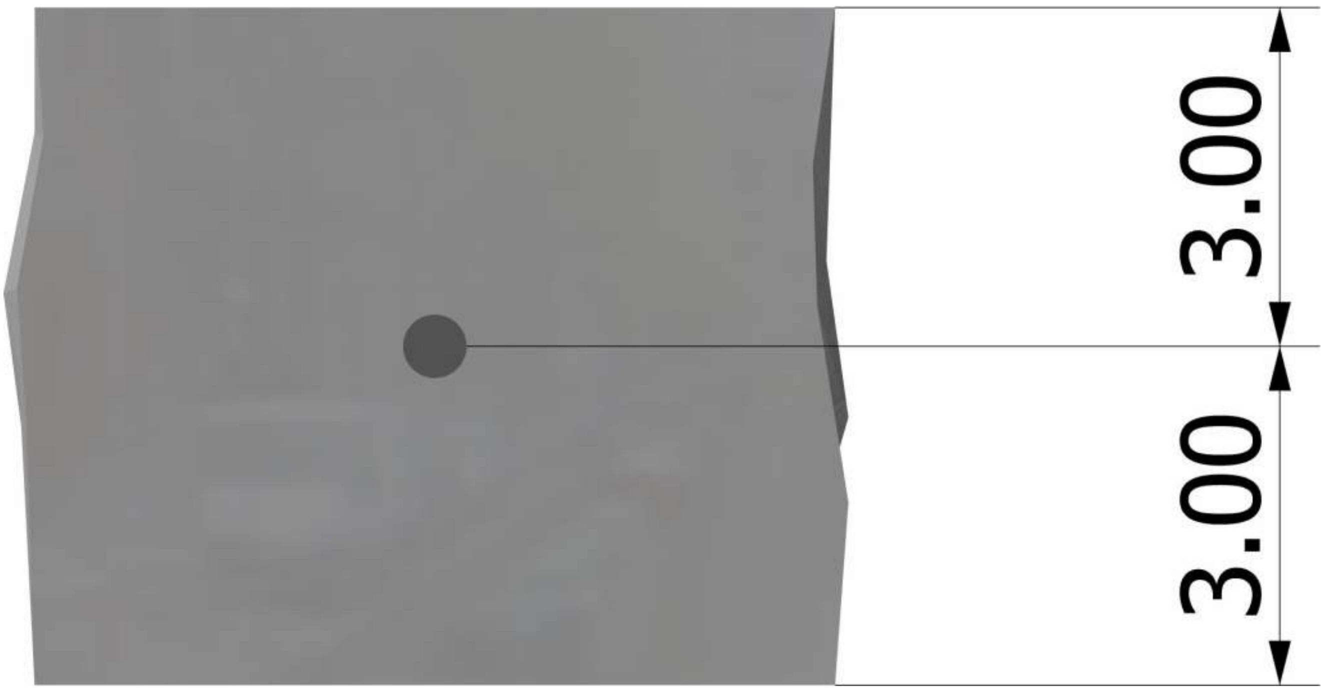
Strength level loads:

N_{ua} [lb]: 3030
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	3030.0	0.0	0.0	0.0
Sum	3030.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 3030
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
8235	0.75	6176

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3500	10.000	44900

$\phi N_{cb} = \phi\ (A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
189.00	900.00	3.00	0.760	1.00	1.000	44900	0.70	5016

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	1.57	3500	0.70	30792

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$\phi N_{sb} = \phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	ϕN_{sb} (lb)
3.00	99999.00	1.57	1.00	3500	0.70	24915

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	3030	6176	0.49	Pass
Concrete breakout	**3030**	**5016**	**0.60**	**Pass (Governs)**
Pullout	3030	30792	0.10	Pass
Side-face blowout	3030	24915	0.12	Pass

PAB4 (1/2"Ø) with hef = 10.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie®	Anchor Designer™ Software Version 2.9.7376.0

Company:	NKH Engineering	Date:	1/25/2021
Engineer:	A.Rishel	Page:	1/5
Project:	Gass Apartment		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: HDU4 Anchor
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.13
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'c (psi): 3500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

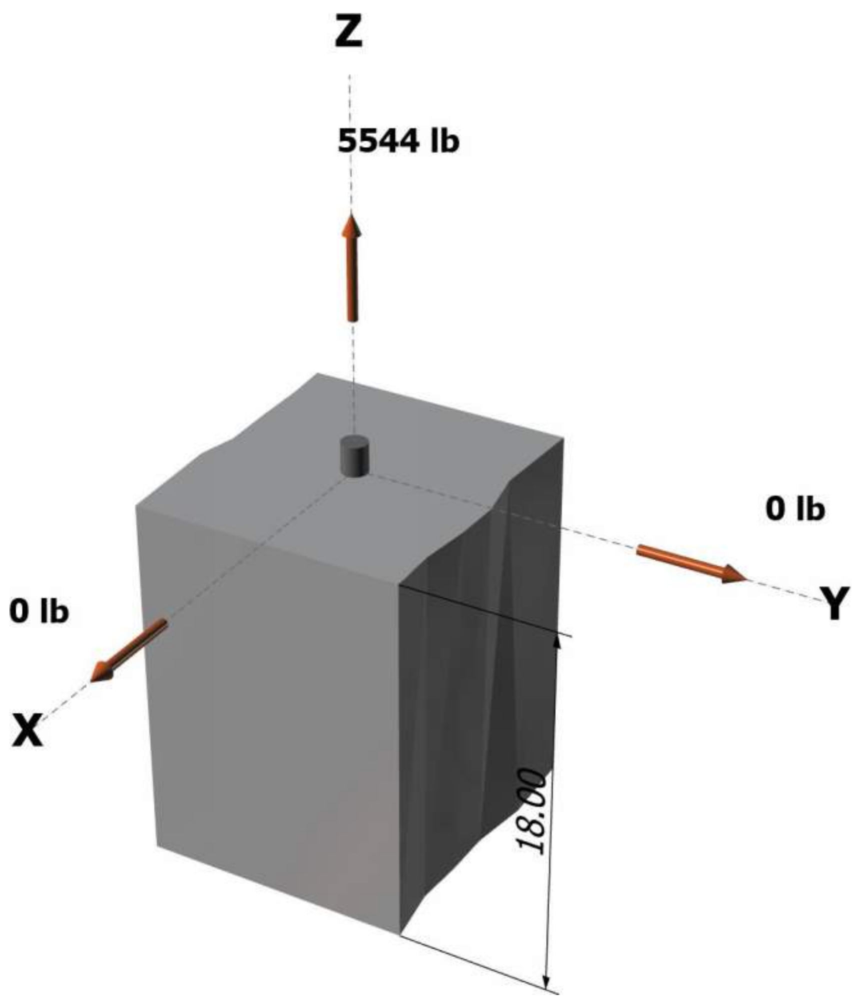
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB5 (5/8"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

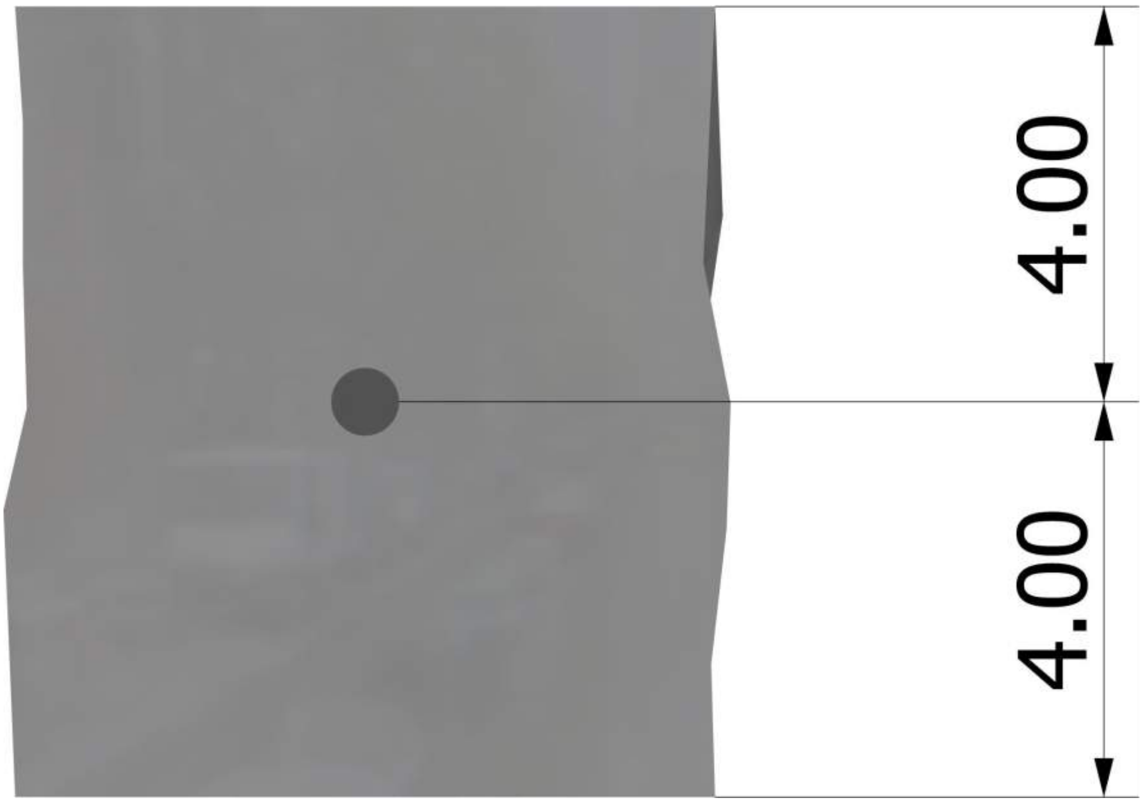
Strength level loads:

N_{ua} [lb]: 5544
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	5544.0	0.0	0.0	0.0
Sum	5544.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 5544
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	3500	12.000	59537

$\phi N_{cb} = \phi\,(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
302.00	1296.00	4.00	0.767	1.00	1.000	59537	0.70	7446

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi\Psi_{c,P}N_p = \phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	2.10	3500	0.70	41121

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$\phi N_{sb} = \phi\{(1+c_{a2}/c_{a1})/4\}(160 c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	ϕN_{sb} (lb)
4.00	99999.00	2.10	1.00	3500	0.70	38390

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	5544	9825	0.56	Pass
Concrete breakout	**5544**	**7446**	**0.74**	**Pass (Governs)**
Pullout	5544	41121	0.13	Pass
Side-face blowout	5544	38390	0.14	Pass

PAB5 (5/8"Ø) with hef = 12.000 inch meets the selected design criteria.

12. Warnings

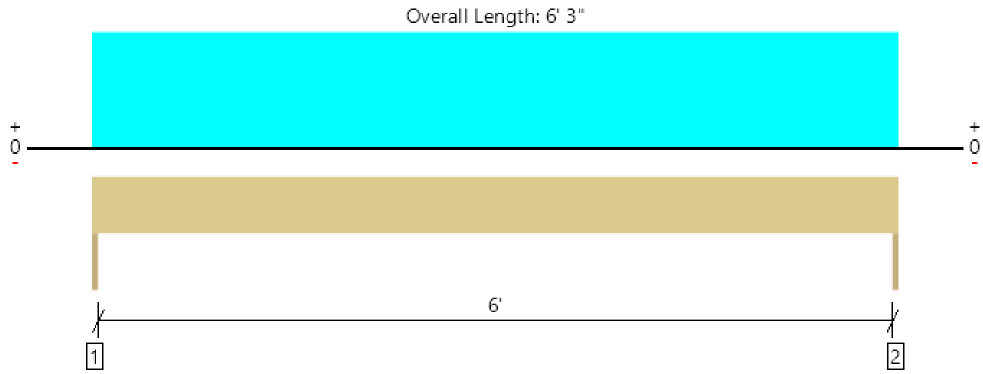
- Designer must exercise own judgement to determine if this design is suitable.

Roof			
Member Name	Results	Current Solution	Comments
Typ Header	Passed	2 piece(s) 2 x 8 DF No.2	
Upper Floor			
Member Name	Results	Current Solution	Comments
Dropped Beam at Walkway	Passed	1 piece(s) 4 x 8 DF No.2	
Garage Header	Passed	1 piece(s) 5 1/2" x 10 1/2" 24F-V4 DF Glulam	
Typ Header	Passed	2 piece(s) 2 x 8 DF No.2	
Existing Roof Joist @ New Deck	Passed	1 piece(s) 2 x 12 DF No.2 @ 16" OC	
Cantilevered Floor Edge Girders	Passed	1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam	
Cantilevered Floor Center Girder	Failed 4	1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam The CCQ66SDS2.5 along with the MSTI72 strap over the beam to the post resist the uplift at the support ∴ approved by EOR	An excessive uplift of -10194 lbs at support located at 14' 8" failed this product.
Cantilevered Floor Beam @ Deck	Passed	1 piece(s) 5 1/2" x 9" 24F-V4 DF Glulam	
Main Floor			
Member Name	Results	Current Solution	Comments
Typ Joist	Passed	1 piece(s) 2 x 10 DF No.2 @ 16" OC	
Typ Drop Beam	Passed	1 piece(s) 6 x 10 DF No.2	

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

9/27/2023 1:04:34 AM UTC
ForteWEB v3.6
File Name: 20-172 Gass Apartment

Roof, Typ Header
2 piece(s) 2 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1580 @ 0	2813 (1.50")	Passed (56%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1211 @ 8 3/4"	3002	Passed (40%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2468 @ 3' 1 1/2"	2720	Passed (91%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.070 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.114 @ 3' 1 1/2"	0.313	Passed (L/659)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) 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"	603	781	977	1580	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	603	781	977	1580	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 3" o/c	
Bottom Edge (Lu)	6' 3" o/c	

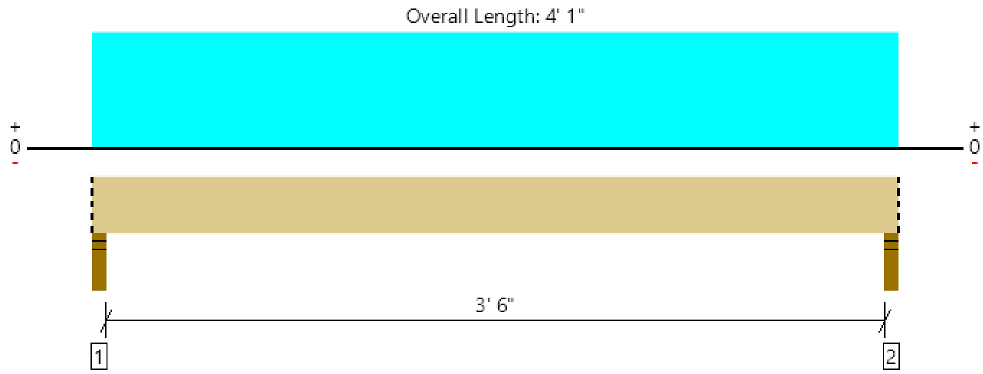
•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	5.5	--	--	
1 - Uniform (PSF)	0 to 6' 3"	12' 6"	15.0	20.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

Upper Floor, Dropped Beam at Walkway
1 piece(s) 4 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2484 @ 2"	7656 (3.50")	Passed (32%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1394 @ 10 3/4"	3045	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2138 @ 2' 1/2"	2989	Passed (72%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.022 @ 2' 1/2"	0.094	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.030 @ 2' 1/2"	0.188	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
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 - Stud wall - DF	3.50"	3.50"	1.50"	687	1797	2484	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	687	1797	2484	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)	4' 1" o/c	
Bottom Edge (Lu)	4' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

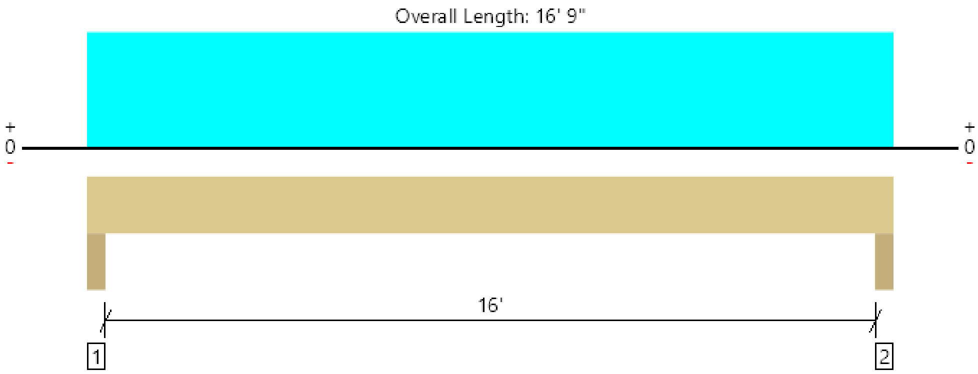
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 4' 1"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 4' 1" (Front)	22'	15.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

Upper Floor, Garage Header

1 piece(s) 5 1/2" x 10 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4179 @ 3"	16088 (4.50")	Passed (26%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3556 @ 1' 3"	10203	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	16472 @ 8' 4 1/2"	20213	Passed (81%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.493 @ 8' 4 1/2"	0.542	Passed (L/396)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.820 @ 8' 4 1/2"	0.813	Passed (L/238)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
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 = 16' 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	Roof Live	Snow	Factored	
1 - Trimmer - DF	4.50"	4.50"	1.50"	1667	2513	168	209	4179	None
2 - Trimmer - DF	4.50"	4.50"	1.50"	1667	2513	168	209	4179	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16' 9" o/c	
Bottom Edge (Lu)	16' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

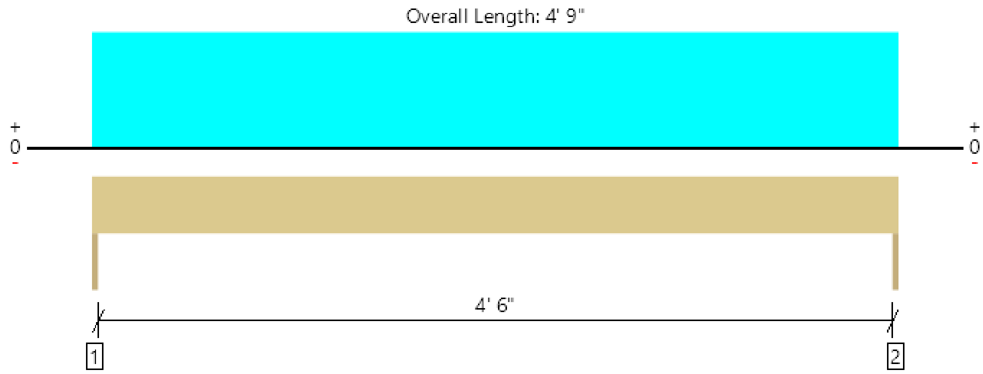
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 16' 9"	N/A	14.0	--	--	--	
1 - Uniform (PSF)	0 to 16' 9"	3'	15.0	40.0	-	-	Floor
2 - Uniform (PSF)	0 to 16' 9"	3'	15.0	60.0	-	-	Balcony
3 - Uniform (PSF)	0 to 16' 9"	1'	15.0	-	20.0	25.0	Roof
4 - Uniform (PSF)	0 to 16' 9"	8'	10.0	-	-	-	Wall

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

MEMBER REPORT

PASSED

Upper Floor, Typ Header
2 piece(s) 2 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1509 @ 0	2813 (1.50")	Passed (54%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1046 @ 8 3/4"	3002	Passed (35%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1792 @ 2' 4 1/2"	2720	Passed (66%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.023 @ 2' 4 1/2"	0.158	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.048 @ 2' 4 1/2"	0.237	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) 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	Roof Live	Snow	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	767	190	594	742	1509	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	767	190	594	742	1509	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 4' 9"	N/A	5.5	--	--	--	
1 - Uniform (PSF)	0 to 4' 9"	12' 6"	15.0	-	20.0	25.0	Roof
2 - Uniform (PSF)	0 to 4' 9"	2'	15.0	40.0	-	-	Floor
3 - Uniform (PSF)	0 to 4' 9"	10'	10.0	-	-	-	Wall

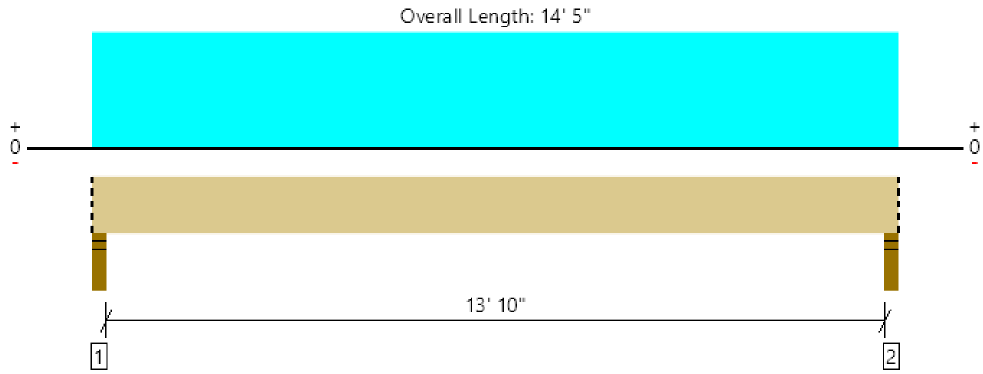
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

MEMBER REPORT

PASSED

Upper Floor, Existing Roof Joist @ New Deck
1 piece(s) 2 x 12 DF No.2 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	721 @ 2 1/2"	3281 (3.50")	Passed (22%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	598 @ 1' 2 3/4"	2025	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2450 @ 7' 2 1/2"	2729	Passed (90%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.243 @ 7' 2 1/2"	0.350	Passed (L/692)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.304 @ 7' 2 1/2"	0.700	Passed (L/553)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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	3.50"	3.50"	1.50"	144	577	721	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	144	577	721	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)	4' 3" o/c	
Bottom Edge (Lu)	14' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

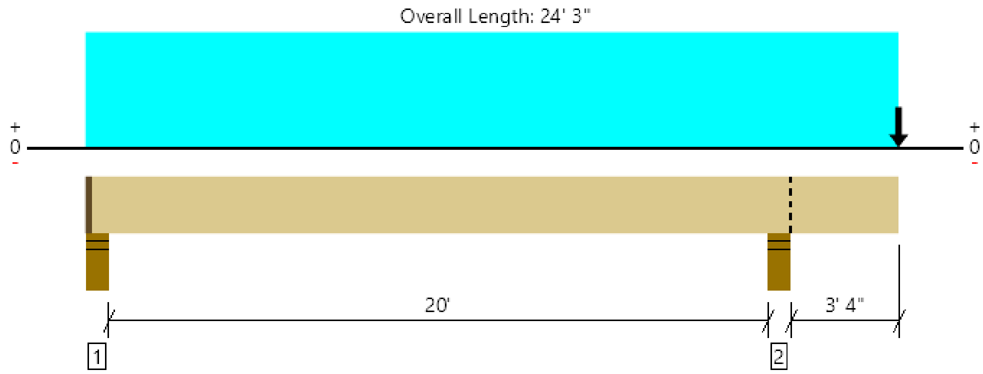
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 14' 5"	16"	15.0	60.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

Upper Floor, Cantilevered Floor Edge Girders

1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	13636 @ 20' 8 1/4"	18906 (5.50")	Passed (72%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	6370 @ 18' 11 1/2"	17490	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	31073 @ 9' 10 3/8"	57189	Passed (54%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-21815 @ 20' 8 1/4"	52260	Passed (42%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.385 @ 10' 6 1/8"	0.509	Passed (L/634)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.466 @ 10' 3 1/4"	1.018	Passed (L/524)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Upward deflection on right cantilever exceeds overhang deflection criteria.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.96 that was calculated using length L = 19' 3/4".
- Critical negative moment adjusted by a volume/size factor of 0.99 that was calculated using length L = 14' 13/16".
- 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	Floor Live	Snow	Factored	
1 - Stud wall - DF	5.50"	4.00"	1.94"	1700	5045/-90	-630	6745	1 1/2" Rim Board
2 - Stud wall - DF	5.50"	5.50"	3.97"	5405	6745	4230	13636	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	24' 2" o/c	
Bottom Edge (Lu)	24' 2" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/2" to 24' 3"	N/A	24.1	--	--	
1 - Uniform (PSF)	0 to 24' 3" (Front)	12'	15.0	40.0	-	Floor Load
2 - Point (lb)	24' 3" (Front)	N/A	2160	-	3600	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

FORTEWEB

MEMBER REPORT

FAILED

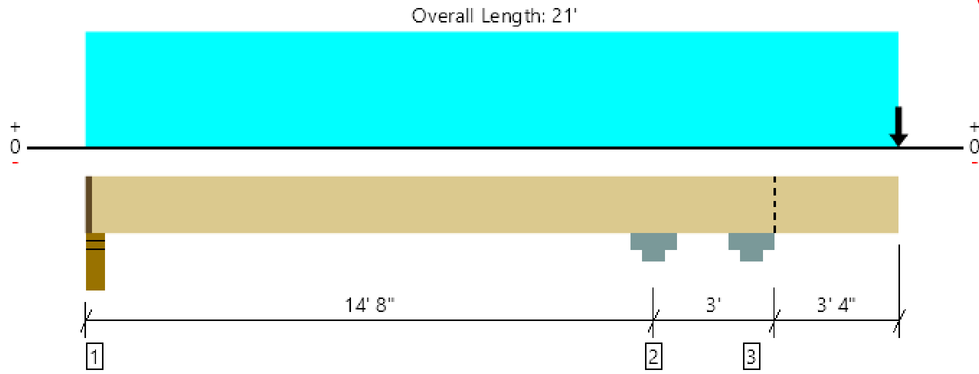
4

The CCQ66SDS2.5 along with the MSTI72 strap over the beam to the post resist the uplift at the support ∴ approved by EOR

Upper Floor, Cantilevered Floor Center Girder

1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam

An excessive uplift of -10194 lbs at support located at 14' 8" failed this product.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	24198 @ 17' 2 1/2"	39325 (11.00")	Passed (62%)	--	1.0 D + 0.75 L + 0.75 S (Adj Spans)
Shear (lbs)	13152 @ 16' 7 1/2"	20114	Passed (65%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	16926 @ 6' 5/16"	59400	Passed (28%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-36694 @ 17' 2 1/2"	52656	Passed (70%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.062 @ 21'	0.200	Passed (2L/999+)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.107 @ 21'	0.379	Passed (2L/848)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/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 = 11' 6 11/16".
- Critical negative moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 8' 1 3/8".
- 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	Snow	Factored	
1 - Stud wall - DF	4.50"	3.00"	1.74"	1816	4294	112	6110	1 1/2" Rim Board
2 - Column Cap - steel	11.00"	11.00"	3.51"	-941	13505/-1260	-9253	12564/-10194	None
3 - Column Cap - steel	11.00"	11.00"	6.77"	8718	5800/-5376	14841	24198	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	20' 11" o/c	
Bottom Edge (Lu)	20' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/2" to 21'	N/A	24.1	--	--	
1 - Uniform (PSF)	0 to 21' (Front)	18'	15.0	40.0	-	Floor Load
2 - Point (lb)	21' (Top)	N/A	3420	-	5700	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

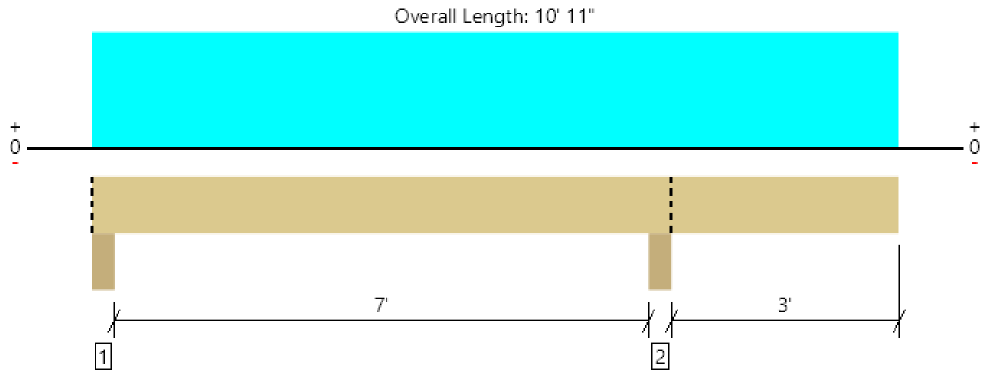
MEMBER REPORT

PASSED

4

Upper Floor, Cantilevered Floor Beam @ Deck

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



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2091 @ 7' 8 1/4"	19663 (5.50")	Passed (11%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	935 @ 6' 8 1/2"	8745	Passed (11%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	1618 @ 3' 9 3/16"	14850	Passed (11%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-1431 @ 7' 8 1/4"	11447	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.028 @ 10' 11"	0.215	Passed (2L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.029 @ 10' 11"	0.323	Passed (2L/999+)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/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 = 6' 10 3/8".
- Critical negative moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 4' 7 3/4".
- 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	Snow	Factored	
1 - Column - DF	5.50"	5.50"	1.50"	312	722/-128	91	1034	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	720	1371	190	2091	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)	10' 11" o/c	
Bottom Edge (Lu)	10' 11" o/c	

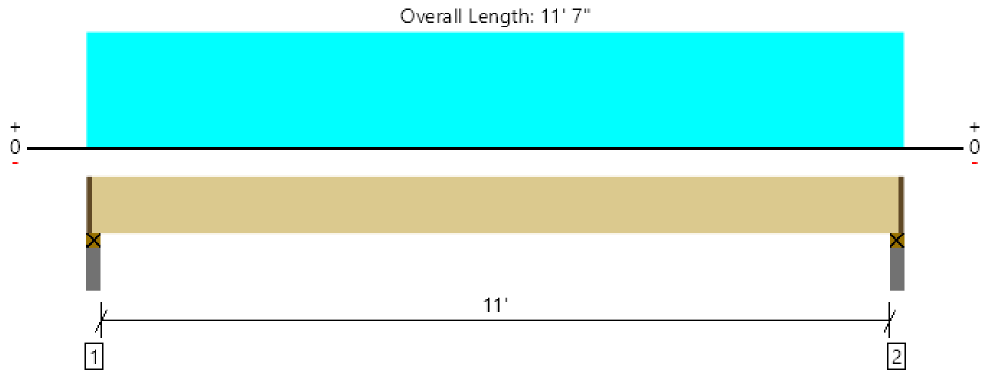
- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 10' 11"	N/A	12.0	--	--	
1 - Uniform (PSF)	0 to 10' 11" (Front)	1'	15.0	-	25.0	Roof Load
2 - Uniform (PSF)	0 to 10' 11" (Front)	4' 6"	15.0	40.0	-	Floor Load

Weyerhaeuser Notes

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

Main Floor, Typ Joist
1 piece(s) 2 x 10 DF No.2 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	493 @ 2 1/2"	2109 (2.25")	Passed (23%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	410 @ 1' 3/4"	1665	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1351 @ 5' 9 1/2"	2029	Passed (67%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.147 @ 5' 9 1/2"	0.279	Passed (L/909)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.192 @ 5' 9 1/2"	0.558	Passed (L/700)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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 - Plate on concrete - DF	3.50"	2.25"	1.50"	116	386	502	1 1/4" Rim Board
2 - Plate on concrete - DF	3.50"	2.25"	1.50"	116	386	502	1 1/4" 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)	9' o/c	
Bottom Edge (Lu)	11' 5" o/c	

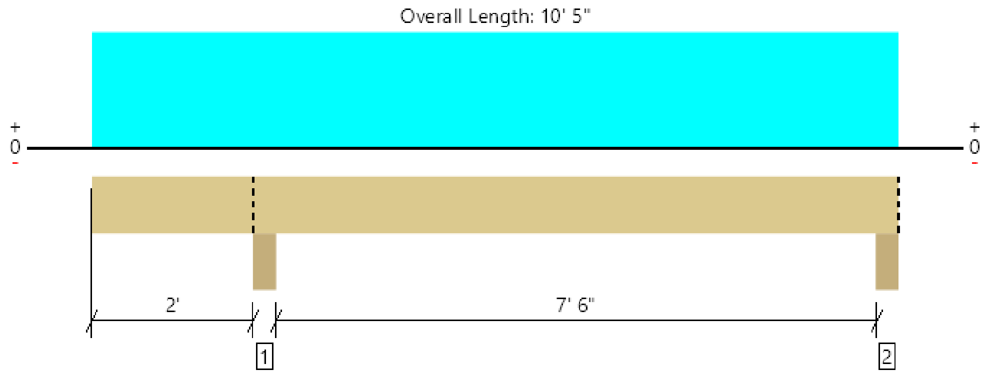
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 11' 7"	16"	15.0	50.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

Main Floor, Typ Drop Beam
1 piece(s) 6 x 10 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4714 @ 2' 2 3/4"	18906 (5.50")	Passed (25%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2347 @ 3' 3"	5922	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5396 @ 6' 2 13/16"	6032	Passed (89%)	1.00	1.0 D + 1.0 L (Alt Spans)
Live Load Defl. (in)	0.092 @ 6' 1 7/8"	0.262	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.116 @ 6' 2 3/16"	0.393	Passed (L/810)	--	1.0 D + 1.0 L (Alt Spans)

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - DF	5.50"	5.50"	1.50"	1154	3560	4714	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	703	2343/-174	3046	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)	10' 5" o/c	
Bottom Edge (Lu)	10' 5" 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)	0 to 10' 5"	N/A	13.2	--	
1 - Uniform (PSF)	0 to 10' 5" (Top)	11'	15.0	50.0	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
Nabil Kausal-Hayes NKH Engineering (206) 601-9728 nabil@nkhengineering.com	

Wood Column

Lic. # : KW-06012717

DESCRIPTION: Walk Way Posts

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	6x6
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	12 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	No.2		
Fb +	900 psi	Fv	180 psi
Fb -	900 psi	Ft	575 psi
Fc - Prll	1350 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1600	1600	1600 ksi
Minimum	580	580	

Exact Width	5.50 in
Exact Depth	5.50 in
Area	30.250 in^2
Ix	76.255 in^4
Iy	76.255 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 10 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 78.675 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.0 ft, D = 2.40, L = 2.320, S = 3.330 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.2712 : 1
	Load Combination	+D+0.750L+0.750S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	6.716 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	818.78 psi
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	+0.60D
	Location of max.above base	12.0 ft
	Applied Design Shear	0.0 psi
	Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y	0.0 in	at	0.0 ft	above base
for load combination	n/a			
Along X-X	0.0 in	at	0.0 ft	above base
for load combination	n/a			

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.621	0.1086	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+L	1.000	0.581	0.2023	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+S	1.150	0.527	0.2345	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+0.750L	1.250	0.496	0.1666	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+0.750L+0.750S	1.150	0.527	0.2712	PASS	0.0 ft	0.0	PASS	12.0 ft
+0.60D	1.600	0.408	0.05584	PASS	0.0 ft	0.0	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					2.479				

Wood Column

DESCRIPTION: Walk Way Posts

Maximum Reactions

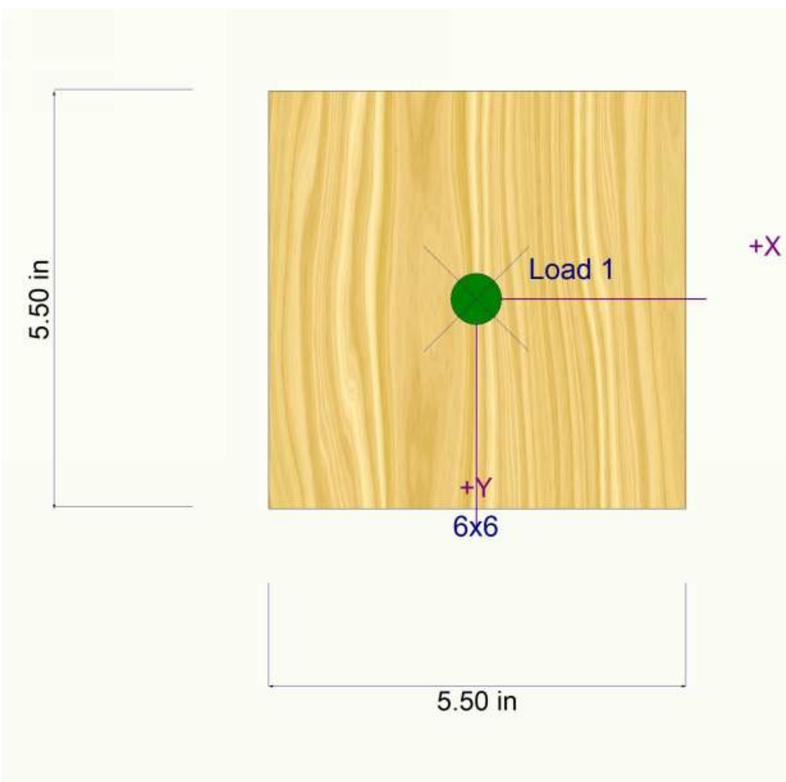
Note: Only non-zero reactions are listed

Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+L					4.799				
+D+S					5.809				
+D+0.750L					4.219				
+D+0.750L+0.750S					6.716				
+0.60D					1.487				
L Only					2.320				
S Only					3.330				

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750L+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft

Sketches



Wood Column

Lic. # : KW-06012717

DESCRIPTION: Typ Stud

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	**2x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	14 ft	Wood Member Type	Sawn
(Used for non-slender calculations)		Exact Width	**1.50** in
Wood Species	Douglas Fir - Larch	Exact Depth	**5.50** in
Wood Grade	No.2	Area	8.250 in^2
		Ix	20.797 in^4
		Iy	1.547 in^4

Fb +	900 psi	Fv	180 psi
Fb -	900 psi	Ft	575 psi
Fc - Prll	1350 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1600	1600	1600 ksi
Minimum	580	580	

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 25.033 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 14.0 ft, D = 0.40, Lr = 0.3320, L = 0.0530, S = 0.4160 k
BENDING LOADS . . .
Wind: Lat. Uniform Load creating Mx-x, W = 0.0530 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.6608** : 1
	Load Combination	+D+0.60W
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	6.953 ft
	At maximum location values are . . .	
	Applied Axial	0.4250 k
	Applied Mx	0.7791 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	2,376.0 psi
PASS	Maximum Shear Stress Ratio =	**0.1405** : 1
	Load Combination	+D+0.60W
	Location of max.above base	14.0 ft
	Applied Design Shear	40.473 psi
	Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.3710 k	Bottom along Y-Y	0.3710 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 1.392 in at 7.047 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	1.000	0.03855	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+L	1.000	1.000	0.03902	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+Lr	1.250	1.000	0.04943	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+S	1.150	1.000	0.05970	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+0.750Lr+0.750L	1.250	1.000	0.04661	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+0.750L+0.750S	1.150	1.000	0.05513	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+0.60W	1.600	1.000	0.6608	PASS	6.953 ft	0.1405	PASS	14.0 ft
+D+0.750Lr+0.750L+0.450W	1.600	1.000	0.4966	PASS	6.953 ft	0.1054	PASS	14.0 ft
+D+0.750L+0.750S+0.450W	1.600	1.000	0.4968	PASS	6.953 ft	0.1054	PASS	14.0 ft

Wood Column

DESCRIPTION: Typ Stud

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.60W	1.600	1.000	0.6605	PASS	6.953 ft	0.1405	PASS	14.0 ft
+0.60D	1.600	1.000	0.01301	PASS	0.0 ft	0.0	PASS	14.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.425				
+D+L					0.478				
+D+Lr					0.757				
+D+S					0.841				
+D+0.750Lr+0.750L					0.714				
+D+0.750L+0.750S					0.777				
+D+0.60W			0.223	0.223	0.425				
+D+0.750Lr+0.750L+0.450W			0.167	0.167	0.714				
+D+0.750L+0.750S+0.450W			0.167	0.167	0.777				
+0.60D+0.60W			0.223	0.223	0.255				
+0.60D					0.255				
Lr Only					0.332				
L Only					0.053				
S Only					0.416				
W Only			0.371	0.371					

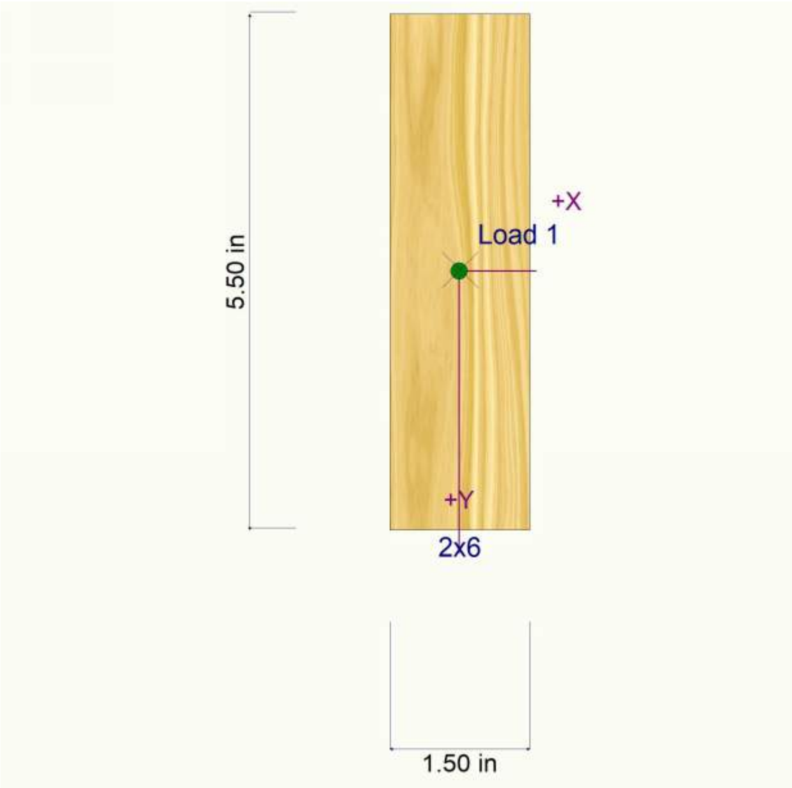
Maximum Deflections for Load Combinations

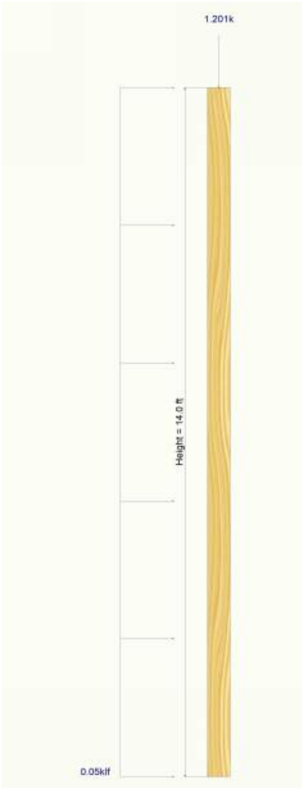
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr+0.750L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750L+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.8350 in	7.047 ft
+D+0.750Lr+0.750L+0.450W	0.0000 in	0.000 ft	0.6262 in	7.047 ft
+D+0.750L+0.750S+0.450W	0.0000 in	0.000 ft	0.6262 in	7.047 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.8350 in	7.047 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
W Only	0.0000 in	0.000 ft	1.3916 in	7.047 ft

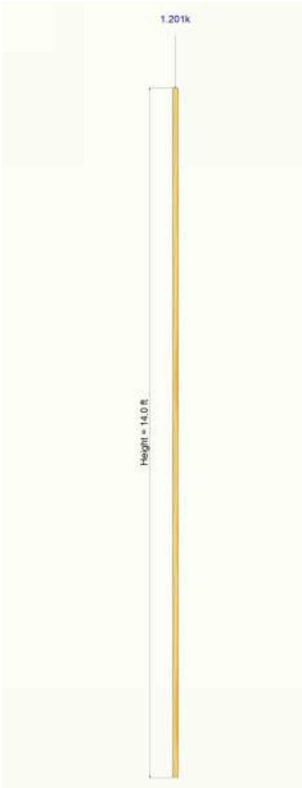
Wood Column

DESCRIPTION: Typ Stud

Sketches







Wood Column

Lic. # : KW-06012717

DESCRIPTION: Trimmer @ Garage Header

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	**3-2x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	10 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch	Exact Width	**4.50** in
Wood Grade	No.2	Exact Depth	**5.50** in
Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

Area	24.750	in^2
Ix	62.391	in^4
Iy	41.766	in^4

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 53.642 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.670, Lr = 0.1680, L = 2.50, S = 0.210 k
BENDING LOADS . . .
Wind: Lat. Uniform Load creating Mx-x, W = 0.3530 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7489** : 1

Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	4.966 ft
At maximum location values are . . .	
Applied Axial	1.724 k
Applied Mx	2.647 k-ft
Applied My	0.0 k-ft
Fc : Allowable	2,376.0 psi

PASS Maximum Shear Stress Ratio = **0.2229** : 1

Load Combination	+D+0.60W
Location of max.above base	10.0 ft
Applied Design Shear	64.182 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.765 k	Bottom along Y-Y	1.765 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.8042 in at 5.034 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	1.000	0.05211	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+L	1.000	1.000	0.1149	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+Lr	1.250	1.000	0.04117	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	1.000	0.04575	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750Lr+0.750L	1.250	1.000	0.08107	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750L+0.750S	1.150	1.000	0.08887	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.60W	1.600	1.000	0.7489	PASS	4.966 ft	0.2229	PASS	10.0 ft
+D+0.750Lr+0.750L+0.450W	1.600	1.000	0.5650	PASS	4.966 ft	0.1671	PASS	10.0 ft
+D+0.750L+0.750S+0.450W	1.600	1.000	0.5651	PASS	4.966 ft	0.1671	PASS	10.0 ft

Wood Column

Lic. # : KW-06012717

DESCRIPTION: Trimmer @ Garage Header

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.60W	1.600	1.000	0.7483	PASS	4.966 ft	0.2229	PASS	10.0 ft
+0.60D	1.600	1.000	0.01759	PASS	0.0 ft	0.0	PASS	10.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					1.724				
+D+L					4.224				
+D+Lr					1.892				
+D+S					1.934				
+D+0.750Lr+0.750L					3.725				
+D+0.750L+0.750S					3.756				
+D+0.60W			1.059	1.059	1.724				
+D+0.750Lr+0.750L+0.450W			0.794	0.794	3.725				
+D+0.750L+0.750S+0.450W			0.794	0.794	3.756				
+0.60D+0.60W			1.059	1.059	1.034				
+0.60D					1.034				
Lr Only					0.168				
L Only					2.500				
S Only					0.210				
W Only			1.765	1.765					

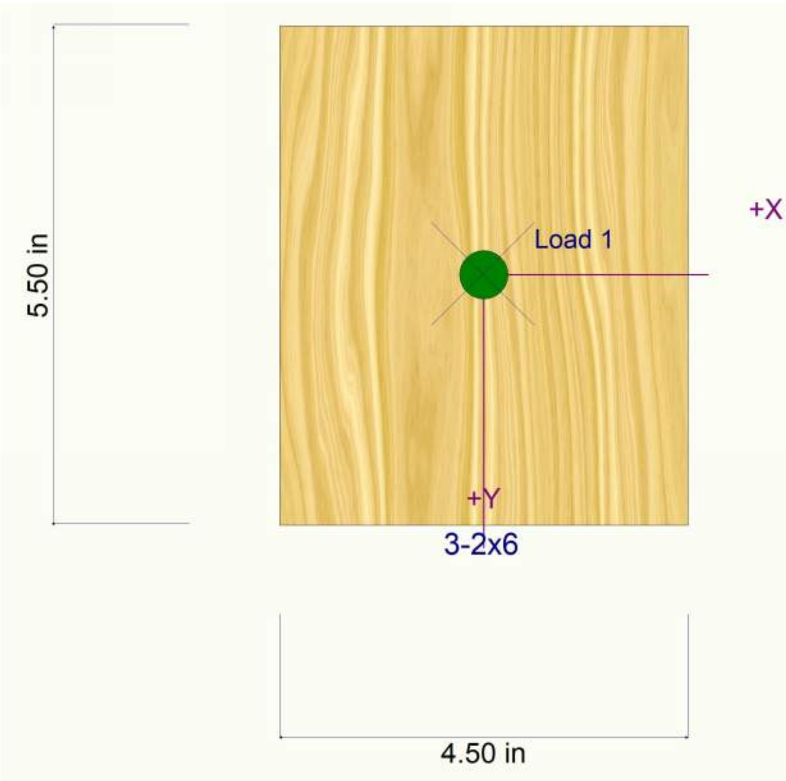
Maximum Deflections for Load Combinations

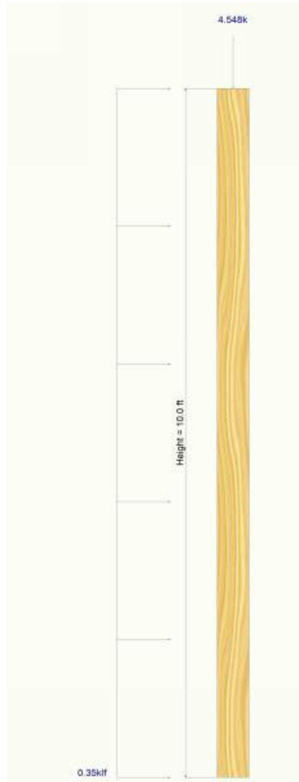
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr+0.750L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750L+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.4825 in	5.034 ft
+D+0.750Lr+0.750L+0.450W	0.0000 in	0.000 ft	0.3619 in	5.034 ft
+D+0.750L+0.750S+0.450W	0.0000 in	0.000 ft	0.3619 in	5.034 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.4825 in	5.034 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.8042 in	5.034 ft

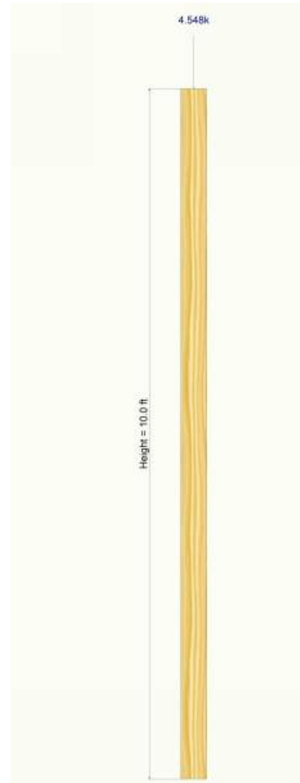
Wood Column

DESCRIPTION: Trimmer @ Garage Header

Sketches







Wood Column

DESCRIPTION: Typ Trimmer

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height: 10 ft
(Used for non-slender calculations)
Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name: 2-2x6
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	3.0 in
Exact Depth	5.50 in
Area	16.50 in^2
Ix	41.594 in^4
Iy	12.375 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 35.761 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 0.80, Lr = 0.670, L = 0.1060, S = 0.8320 k
BENDING LOADS . . .
Wind: Lat. Uniform Load creating Mx-x, W = 0.1060 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3374** : 1

Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	4.966 ft
At maximum location values are . . .	
Applied Axial	0.8358 k
Applied Mx	0.7950 k-ft
Applied My	0.0 k-ft
Fc : Allowable	2,376.0 psi

PASS Maximum Shear Stress Ratio = **0.1004** : 1

Load Combination	+D+0.60W
Location of max.above base	10.0 ft
Applied Design Shear	28.909 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.530 k	Bottom along Y-Y	0.530 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.3622 in at 5.034 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	1.000	0.03790	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+L	1.000	1.000	0.03844	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+Lr	1.250	1.000	0.04916	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	1.000	0.05919	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750Lr+0.750L	1.250	1.000	0.04629	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750L+0.750S	1.150	1.000	0.05463	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.60W	1.600	1.000	0.3374	PASS	4.966 ft	0.1004	PASS	10.0 ft
+D+0.750Lr+0.750L+0.450W	1.600	1.000	0.2540	PASS	5.034 ft	0.07528	PASS	10.0 ft
+D+0.750L+0.750S+0.450W	1.600	1.000	0.2542	PASS	5.034 ft	0.07528	PASS	10.0 ft

Wood Column

DESCRIPTION: Typ Trimmer

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.60W	1.600	1.000	0.3371	PASS	4.966 ft	0.1004	PASS	10.0 ft
+0.60D	1.600	1.000	0.01279	PASS	0.0 ft	0.0	PASS	10.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.836				
+D+L					0.942				
+D+Lr					1.506				
+D+S					1.668				
+D+0.750Lr+0.750L					1.418				
+D+0.750L+0.750S					1.539				
+D+0.60W			0.318	0.318	0.836				
+D+0.750Lr+0.750L+0.450W			0.239	0.239	1.418				
+D+0.750L+0.750S+0.450W			0.239	0.239	1.539				
+0.60D+0.60W			0.318	0.318	0.501				
+0.60D					0.501				
Lr Only					0.670				
L Only					0.106				
S Only					0.832				
W Only			0.530	0.530					

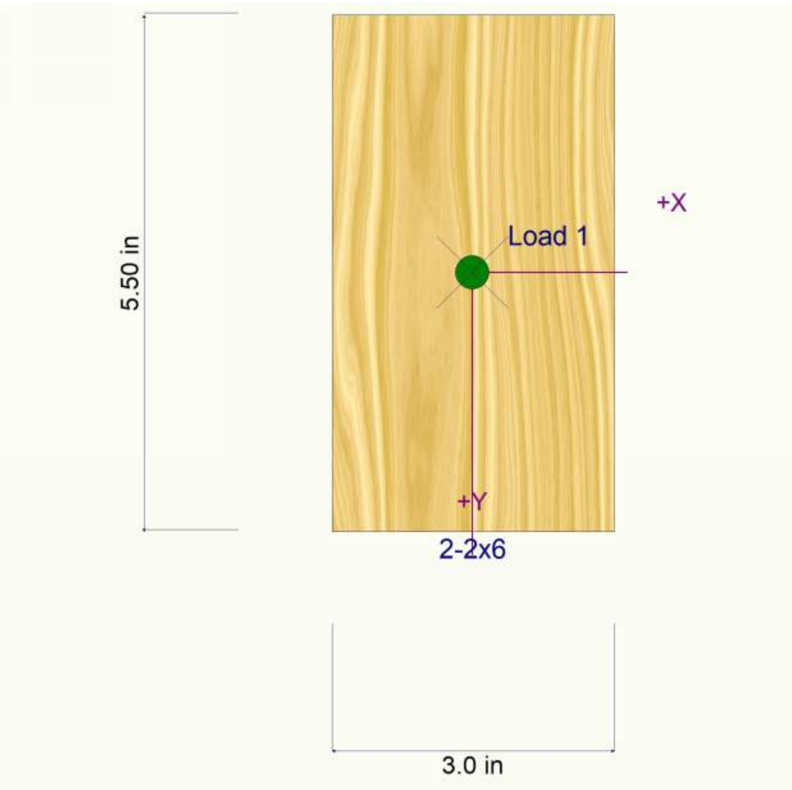
Maximum Deflections for Load Combinations

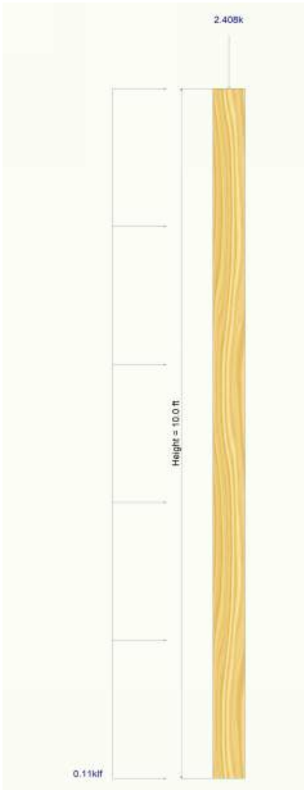
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr+0.750L	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750L+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.2173 in	5.034 ft
+D+0.750Lr+0.750L+0.450W	0.0000 in	0.000 ft	0.1630 in	5.034 ft
+D+0.750L+0.750S+0.450W	0.0000 in	0.000 ft	0.1630 in	5.034 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.2173 in	5.034 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.3622 in	5.034 ft

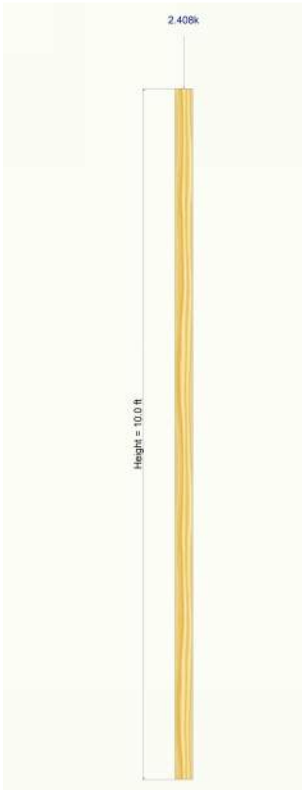
Wood Column

DESCRIPTION: Typ Trimmer

Sketches







Wall Footing

Lic. # : KW-06012717

DESCRIPTION: Typ Wall Footing

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
AutoCalc Footing Weight as DL	:	Yes	

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Adjusted Allowable Bearing Pressure = 1.50 ksf

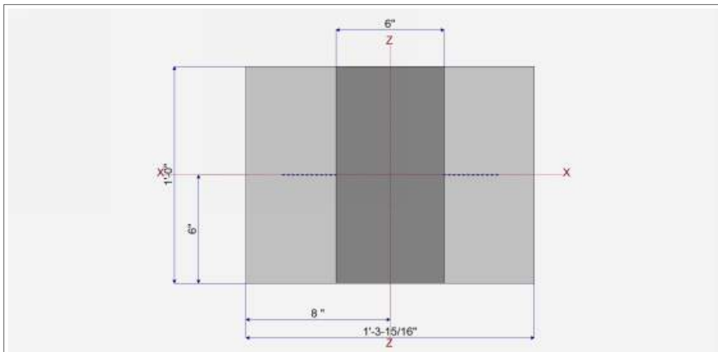
Dimensions

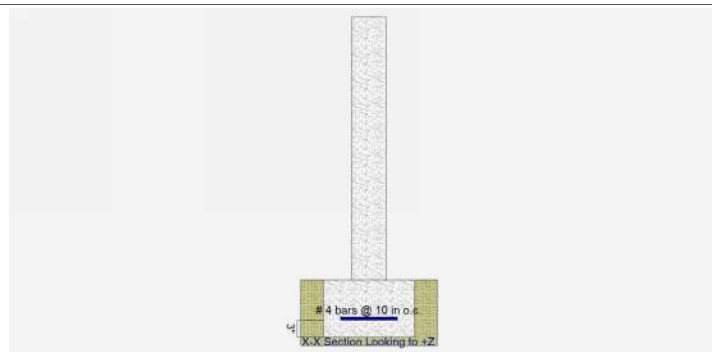
Footing Width	=	1.33	ft
Wall Thickness	=	6.0	in
Wall center offset from center of footing	=	0	in
Footing Thickness	=	10.0	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0	in

Reinforcing

Bars along X-X Axis

Bar spacing	=	10.00
Reinforcing Bar Size	=	# 4





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	0.440	0.250	0.040	0.3130				k
OB : Overburden	=								ksf
V-x	=								k
M-zz	=								k-ft

Vx applied = in above top of footing

Wall Footing

DESCRIPTION: Typ Wall Footing

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4580	Soil Bearing	0.6870 ksf	1.50 ksf	+D+S
PASS	0.01108	Z Flexure (+X)	0.08037 k-ft	7.255 k-ft	+1.20D+0.50L+1.60S
PASS	0.004823	Z Flexure (-X)	0.03499 k-ft	7.255 k-ft	+0.90D
PASS	n/a	1-way Shear (+X)	0.0 psi	75.0 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress -X	Actual Soil Bearing Stress +X	Actual / Allowable Ratio
, D Only	1.50 ksf	0.0 in	0.4517 ksf	0.4517 ksf	0.301
, +D+L	1.50 ksf	0.0 in	0.4817 ksf	0.4817 ksf	0.321
, +D+Lr	1.50 ksf	0.0 in	0.6396 ksf	0.6396 ksf	0.426
, +D+S	1.50 ksf	0.0 in	0.6870 ksf	0.6870 ksf	0.458
, +D+0.750Lr+0.750L	1.50 ksf	0.0 in	0.6152 ksf	0.6152 ksf	0.410
, +D+0.750L+0.750S	1.50 ksf	0.0 in	0.6507 ksf	0.6507 ksf	0.434
, +0.60D	1.50 ksf	0.0 in	0.2710 ksf	0.2710 ksf	0.181

Overturning Stability

Units : k-ft

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status

Footing Has NO Sliding

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D	0.05444	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.40D	0.05444	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50Lr+1.60L	0.05889	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50Lr+1.60L	0.05889	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60L+0.50S	0.06093	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60L+0.50S	0.06093	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60Lr+0.50L	0.07384	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60Lr+0.50L	0.07384	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60Lr	0.07255	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60Lr	0.07255	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+1.60S	0.08037	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+1.60S	0.08037	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60S	0.07907	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+1.60S	0.07907	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50Lr+0.50L	0.05604	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50Lr+0.50L	0.05604	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+0.50S	0.05808	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+0.50S	0.05808	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+0.20S	0.05201	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +1.20D+0.50L+0.20S	0.05201	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +0.90D	0.03499	-X	Bottom	0.216	Min Temp %	0.24	7.255	OK
, +0.90D	0.03499	+X	Bottom	0.216	Min Temp %	0.24	7.255	OK

Wall Footing

Lic. # : KW-06012717

DESCRIPTION: Typ Wall Footing

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0 psi	0 psi	0 psi	75 psi	0	OK

Wall Footing

Lic. # : KW-06012717

DESCRIPTION: Typ Wall Footing

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.20D+0.50Lr+1.60L	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+1.60L+0.50S	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+1.60Lr+0.50L	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+1.60Lr	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+0.50L+1.60S	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+1.60S	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+0.50Lr+0.50L	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+0.50L+0.50S	0 psi	0 psi	0 psi	75 psi	0	OK
+1.20D+0.50L+0.20S	0 psi	0 psi	0 psi	75 psi	0	OK
+0.90D	0 psi	0 psi	0 psi	75 psi	0	OK

General Footing

Lic. # : KW-06012717

DESCRIPTION: Footing - Walkway Post

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

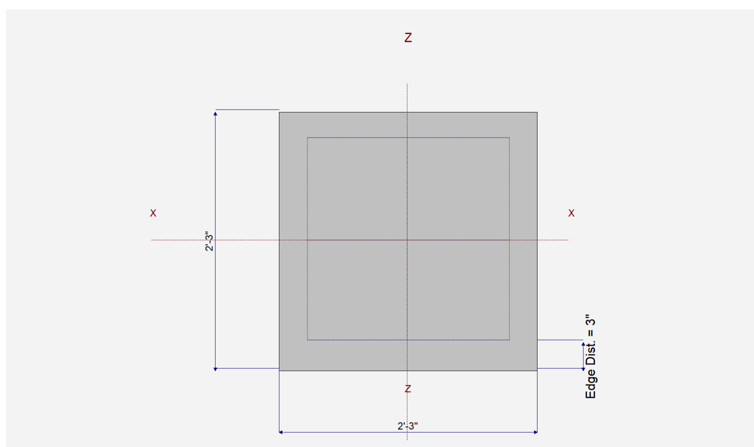
Footing base depth below soil surface	=	1.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	2.25	ft
Length parallel to Z-Z Axis	=	2.250	ft
Footing Thickness	=	10.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

Number of Bars	=		3
Reinforcing Bar Size	=	#	4

Bars parallel to Z-Z Axis

Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation n/a

Bars required within zone n/a

Bars required on each side of zone n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.40		2.320	3.330				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Lic. # : KW-06012717

DESCRIPTION: Footing - Walkway Post

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9667	Soil Bearing	1.450 ksf	1.50 ksf	+D+0.750L+0.750S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1448	Z Flexure (+X)	1.171 k-ft/ft	8.086 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.1448	Z Flexure (-X)	1.171 k-ft/ft	8.086 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.1448	X Flexure (+Z)	1.171 k-ft/ft	8.086 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.1448	X Flexure (-Z)	1.171 k-ft/ft	8.086 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.1448	1-way Shear (+X)	11.896 psi	82.158 psi	+1.20D+0.50L+1.60S
PASS	0.1448	1-way Shear (-X)	11.896 psi	82.158 psi	+1.20D+0.50L+1.60S
PASS	0.1448	1-way Shear (+Z)	11.896 psi	82.158 psi	+1.20D+0.50L+1.60S
PASS	0.1448	1-way Shear (-Z)	11.896 psi	82.158 psi	+1.20D+0.50L+1.60S
PASS	0.2712	2-way Punching	44.565 psi	164.317 psi	+1.20D+0.50L+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.50	n/a	0.0	0.6132	0.6132	n/a	n/a	0.409
X-X, +D+L	1.50	n/a	0.0	1.072	1.072	n/a	n/a	0.715
X-X, +D+S	1.50	n/a	0.0	1.271	1.271	n/a	n/a	0.847
X-X, +D+0.750L	1.50	n/a	0.0	0.9569	0.9569	n/a	n/a	0.638
X-X, +D+0.750L+0.750S	1.50	n/a	0.0	1.450	1.450	n/a	n/a	0.967
X-X, +0.60D	1.50	n/a	0.0	0.3679	0.3679	n/a	n/a	0.245
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.6132	0.6132	0.409
Z-Z, +D+L	1.50	0.0	n/a	n/a	n/a	1.072	1.072	0.715
Z-Z, +D+S	1.50	0.0	n/a	n/a	n/a	1.271	1.271	0.847
Z-Z, +D+0.750L	1.50	0.0	n/a	n/a	n/a	0.9569	0.9569	0.638
Z-Z, +D+0.750L+0.750S	1.50	0.0	n/a	n/a	n/a	1.450	1.450	0.967
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.3679	0.3679	0.245

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status

Footing Has NO Sliding

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.420	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.40D	0.420	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+1.60L	0.8240	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+1.60L	0.8240	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+1.60L+0.50S	1.032	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+1.60L+0.50S	1.032	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L	0.5050	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L	0.5050	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D	0.360	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D	0.360	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+1.60S	1.171	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+1.60S	1.171	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK

General Footing

Lic. # : KW-06012717

DESCRIPTION: Footing - Walkway Post

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.20D+1.60S	1.026	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+1.60S	1.026	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+0.50S	0.7131	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+0.50S	0.7131	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+0.20S	0.5883	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +1.20D+0.50L+0.20S	0.5883	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +0.90D	0.270	+Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
X-X, +0.90D	0.270	-Z	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.40D	0.420	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.40D	0.420	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60L	0.8240	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60L	0.8240	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60L+0.50S	1.032	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60L+0.50S	1.032	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L	0.5050	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L	0.5050	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D	0.360	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D	0.360	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+1.60S	1.171	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+1.60S	1.171	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60S	1.026	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+1.60S	1.026	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+0.50S	0.7131	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+0.50S	0.7131	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+0.20S	0.5883	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +1.20D+0.50L+0.20S	0.5883	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +0.90D	0.270	-X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK
Z-Z, +0.90D	0.270	+X	Bottom	0.2160	Min Temp %	0.2667	8.086	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	4.27 psi	4.27 psi	4.27 psi	4.27 psi	4.27 psi	82.16 psi	0.05	OK
+1.20D+1.60L	8.37 psi	8.37 psi	8.37 psi	8.37 psi	8.37 psi	82.16 psi	0.10	OK
+1.20D+1.60L+0.50S	10.49 psi	10.49 psi	10.49 psi	10.49 psi	10.49 psi	82.16 psi	0.13	OK
+1.20D+0.50L	5.13 psi	5.13 psi	5.13 psi	5.13 psi	5.13 psi	82.16 psi	0.06	OK
+1.20D	3.66 psi	3.66 psi	3.66 psi	3.66 psi	3.66 psi	82.16 psi	0.04	OK
+1.20D+0.50L+1.60S	11.90 psi	11.90 psi	11.90 psi	11.90 psi	11.90 psi	82.16 psi	0.14	OK
+1.20D+1.60S	10.42 psi	10.42 psi	10.42 psi	10.42 psi	10.42 psi	82.16 psi	0.13	OK
+1.20D+0.50L+0.50S	7.24 psi	7.24 psi	7.24 psi	7.24 psi	7.24 psi	82.16 psi	0.09	OK
+1.20D+0.50L+0.20S	5.98 psi	5.98 psi	5.98 psi	5.98 psi	5.98 psi	82.16 psi	0.07	OK
+0.90D	2.74 psi	2.74 psi	2.74 psi	2.74 psi	2.74 psi	82.16 psi	0.03	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	15.98 psi	164.32 psi	0.09728	OK
+1.20D+1.60L	31.36 psi	164.32 psi	0.1908	OK
+1.20D+1.60L+0.50S	39.28 psi	164.32 psi	0.239	OK
+1.20D+0.50L	19.22 psi	164.32 psi	0.117	OK
+1.20D	13.70 psi	164.32 psi	0.08338	OK
+1.20D+0.50L+1.60S	44.57 psi	164.32 psi	0.2712	OK
+1.20D+1.60S	39.05 psi	164.32 psi	0.2376	OK
+1.20D+0.50L+0.50S	27.14 psi	164.32 psi	0.1652	OK
+1.20D+0.50L+0.20S	22.39 psi	164.32 psi	0.1362	OK
+0.90D	10.28 psi	164.32 psi	0.06253	OK