

# MYERS ENGINEERING

## Lateral & Gravity Calculations



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signed by  
Mark  
Myers, PE  
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MUST BEAR ORIGINAL BLUE INK SIGNATURE OR  
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**Project: Long Residence Addition & Remodel**  
**3424 76<sup>th</sup> Place Southeast**  
**Mercer Island, WA**

September 22, 2024

2021 INTERNATIONAL BUILDING CODE  
100 MPH BASIC WIND, EXPOSURE B,  $K_{zt} = 1.00$   
RISK CATEGORY II - SOIL SITE CLASS D  
SEISMIC DESIGN CATEGORY D (IBC)

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**DESIGN LOADS:**

ROOF DEAD LOADS	15 PSF Total
ROOF LIVE LOADS	25 PSF (Snow)
FLOOR DEAD LOADS	15 PSF Total
FLOOR LIVE LOADS	40 PSF (Reducible)
STAIR LIVE LOADS	100 PSF

$$\text{psf} := \frac{\text{lb}}{\text{ft}^2}$$

$$\text{plf} := \frac{\text{lb}}{\text{ft}}$$

**WOODS :**

**WOOD TYPE:**

JOISTS OR RAFTERS 2X.....	HF#2
BEAMS OR HEADERS 4X.....	DF#2
BEAMS OR HEADERS 6X OR LARGER.....	DF#1
LEDGERS AND TOP PLATES.....	HF#2
STUDS 2X4 OR 2X6.....	HF Stud
POSTS	
4X4.....	HF#2
4X6.....	HF#2
6X6.....	DF#1

**GLUED-LAMINATED (GLB) BEAM & HEADER.**

Fb=2,400 PSI, Fv=165 PSI, Fc (Perp) =650 PSI, E=1,800,000 PSI.

**PARALLAM (PSL) 2.0E BEAM & HEADER.**

Fb=2,900 PSI, Fv=290 PSI, Fc (Perp) =750 PSI, E=2,000,000 PSI.

**MICROLAM (LVL) 1.9E BEAM & HEADER**

Fb=2,600 PSI, Fv=285 PSI, Pc (Perp) =750 PSI, E=1,900,000 PSI.

**TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD**

Fb=1,700 PSI, Fv=400 PSI, Pc (Perp) =680 PSI, E=1,300,000 PSI.

**TRUSSES:**

PREFABRICATED WOOD TRUSSES SHALL BE DESIGNED BY A REGISTERED DESIGN PROFESSIONAL REGISTERED IN THE STATE OF WASHINGTON. TRUSS DESIGNS SHALL COMPLY WITH THE REQUIREMENTS OF IBC 2303.4. SUBMITTAL PACKAGE SHALL COMPLY WITH REQUIREMENTS OF IBC 2303.4.1.4.

UNLESS OTHERWISE SPECIFIED BY LOCAL BUILDING OFFICIAL OR STATUTE, TRUSS DESIGNS BEARING THE SEAL AND SIGNATURE OF THE TRUSS DESIGNER SHALL BE AVAILABLE AT TIME OF INSPECTION.

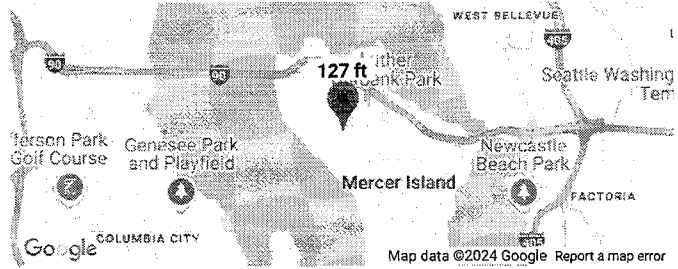
**ENGINEERED I-JOISTS**

-FLOOR JOISTS & BEAMS OF EQUAL OR BETTER CAPACITY MAY BE SUBSTITUTED FOR THOSE SHOWN ON THIS PLAN, "EQUAL" IS DEFINED AS HAVING MOMENT CAPACITY, SHEAR CAPACITY, AND STIFFNESS WITHIN 3% OF THE SPECIFIED JOISTS OR BEAMS.

# ATC Hazards by Location

## Search Information

**Coordinates:** 47.579, -122.237  
**Elevation:** 127 ft  
**Timestamp:** 2024-09-21T00:06:25.794Z  
**Hazard Type:** Seismic  
**Reference Document:** ASCE7-16  
**Risk Category:** II  
**Site Class:** D



## Basic Parameters

Name	Value	Description
$S_S$	1.41	$MCE_R$ ground motion (period=0.2s)
$S_1$	0.49	$MCE_R$ ground motion (period=1.0s)
$S_{MS}$	1.41	Site-modified spectral acceleration value
$S_{M1}$	* null	Site-modified spectral acceleration value
$S_{DS}$	0.94	Numeric seismic design value at 0.2s SA
$S_{D1}$	* null	Numeric seismic design value at 1.0s SA

\* See Section 11.4.8

## Additional Information

Name	Value	Description
SDC	* null	Seismic design category
$F_a$	1	Site amplification factor at 0.2s
$F_v$	* null	Site amplification factor at 1.0s
$CR_S$	0.902	Coefficient of risk (0.2s)
$CR_1$	0.897	Coefficient of risk (1.0s)
PGA	0.603	$MCE_G$ peak ground acceleration
$F_{PGA}$	1.1	Site amplification factor at PGA
$PGA_M$	0.664	Site modified peak ground acceleration
$T_L$	6	Long-period transition period (s)
$SsRT$	1.41	Probabilistic risk-targeted ground motion (0.2s)
$SsUH$	1.562	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
$SsD$	3.446	Factored deterministic acceleration value (0.2s)
$S1RT$	0.49	Probabilistic risk-targeted ground motion (1.0s)
$S1UH$	0.547	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
$S1D$	1.389	Factored deterministic acceleration value (1.0s)
$PGAd$	1.181	Factored deterministic acceleration value (PGA)

\* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

## Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

**LATERAL ANALYSIS :**

BASED ON 2021 INTERNATIONAL BUILDING CODE (IBC)

Lateral Forces will be distributed along lines of Force/Resistance. Lines of Force/Resistance will be investigated for both wind and seismic lateral loads. Roof and Floor diaphragms are considered flexible.

Risk Category II per IBC 1604.5 & Soils Site Class D (Assumed)

**SEISMIC DESIGN:**

SEISMIC DESIGN BASED ON 2021 IBC Section 1613.1

LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT ABOVE GRADE.

**Seismic Design Data:**

$I_e := 1.0$  (ASCE 7-16 Table 1.5-2)

$R_{wv} := 6.5$      $\Omega_0 := 3.0$      $C_d := 4$     Light-frame (wood) walls sheathed w/ wood structural panels rated for shear resistance (ASCE 7-16 Table 12.2-1)

$S_s := 1.41$                        $S_1 := 0.49$                        $S_{ms} := 1.41$                        $S_{m1} := 0.9$

Equation 11.4-3               $S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.94$                       Equation 11.4-4               $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.6$

–Seismic Design Category D ( $S_{DS}$  greater than 0.50g &  $S_{D1}$  greater than 0.20g)

Roof Slope Adjustment Factor:               $S_a := \frac{1}{\cos\left(\text{atan}\left(\frac{1}{12}\right)\right)} = 1$

Plan Area for Each Level:

$A_1 := 2253\text{ft}^2 \cdot S_a$                $A_{2a} := 1132\text{ft}^2$                        $A_{2b} := 324\text{ft}^2$   
 (Upper Roof)                      (Main Floor Tributary)                      (Deck)

Plan Perimeter for Each Level:

$P_1 := 2(54.33\text{ft}) + 2(55.25\text{ft})$                $P_2 := 2(33\text{ft}) + 2(45.25\text{ft})$   
 (Main Floor)                      (Lower Floor Tributary)

$W, w_x$  = Seismic Weight of Overall Structure, Seismic Weight of Structure above Level x (LB.)

Weight of Structure at Each Level:

Story Weight at Upper Floor:

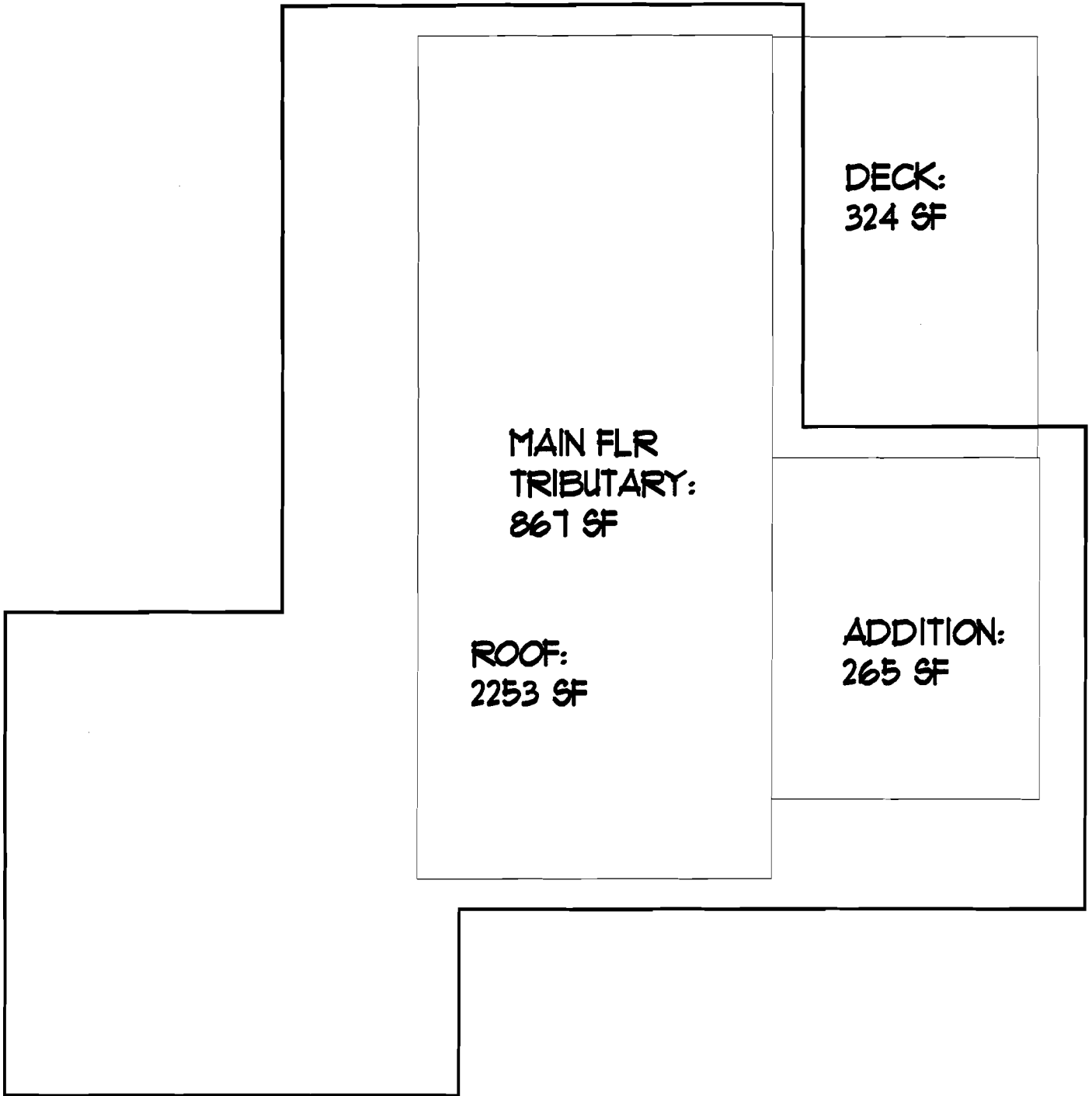
$w_1 := 15 \cdot \text{psf} \cdot A_1 + 12 \cdot \text{psf} \cdot 4.4 \cdot \text{ft} \cdot P_1$

Weight of floors include 10psf weight of floor framing, flooring material, insulation, plus 10psf for miscellaneous interior walls.

Story Weight at Main Floor:

$w_2 := 15 \cdot \text{psf} \cdot (A_{2a}) + 10 \cdot \text{psf} \cdot (A_{2b}) + 12 \cdot \text{psf} \cdot (4.4 \cdot \text{ft} \cdot P_1 + 4 \cdot \text{ft} \cdot P_2)$

$W_{wv} := w_1 + w_2 = 84787.44 \text{ lb}$



Approximate Fundamental Period,  $T_a$ :

$$C_t := 0.02 \quad \chi := 0.75 \quad (\text{per ASCE 7-16 Table 12.8-2}) \quad h_n := 25 \quad (\text{Structural Height per ASCE 7-16 Sect. 11.2})$$

$$T_a := C_t \cdot h_n^\chi = 0.22 \quad (\text{ASCE 7-16 Eq. 12.8-7}) \quad T_L := 6 \quad (\text{per ASCE 7-16 Fig. 22-14})$$

$T_a$  is less than  $T_L$ , therefore  $C_s$  need not exceed:

$$\frac{S_{D1}}{\left(\frac{R}{I_e}\right) \cdot T_a} = 0.41 \quad (\text{ASCE 7-16 Eq. 12.8-3})$$

$C_s$  shall not be less than:  $0.044S_{DS} \cdot I_e = 0.04$  (ASCE 7-16 Eq. 12.8-5)

$$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.14 \quad (\text{ASCE 7-16 Eq. 12.8-2})$$

Total Base Shear:  $V_E := C_s \cdot W = 12261.57 \text{ lb}$

Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

for structures having a period of 0.5 sec or less:  $k := 1$

$h_1 := 18 \text{ ft}$        $h_2 := 8 \text{ ft}$       (Height from base to level x)

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.72 \quad F_1 := C_{v1} \cdot V_E = 8859.16 \text{ lb} \quad \text{Story Shear at Upper Floor}$$

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.28 \quad F_2 := C_{v2} \cdot V_E = 3402.41 \text{ lb} \quad \text{Story Shear at Main Floor}$$

**WIND DESIGN**

Use analytical procedure of ASCE 7-16 Chapter 27 (Directional Procedure for buildings of all heights)

$V_{\text{ww}} := 100$  Nominal 3-Sec Gust (MPH) for Risk Category II (Figure 26.5-1B).

$K_d := 0.85$  Wind Directionality Factor (Table 26.6-1).  $h := 18\text{-ft}$  Mean Roof Height as per Sect. 26.2

$K_e := 1$  Ground Elevation Factor (Sect. 26.9)

Exposure Category B (ASCE 7-16 Sect. 26.7.3)

Topographic Factor ( $K_{zt}$ ) (Figure 26.8-1): 2-D Escarpment with building downwind of crest.

$x := 1\text{ft}$   $\frac{H}{\text{ww}} := 1\text{ft}$   $L_h := 1\text{ft}$   $z := h$   $\gamma := 2.5$   $\mu := 4$

$$K_1 := 0.75 \left( \frac{H}{L_h} \right) = 0.75 \quad K_2 := \left( 1 - \frac{x}{\mu L_h} \right) = 0.75 \quad K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0 \quad K_{zt} := (1 + K_1 \cdot K_2 \cdot K_3)^2 = 1$$

$G_{\text{ww}} := 0.85$  Gust Effect Factor (ASCE 7-16 Sect. 26.11.1)

Building is an Enclosed Building as per ASCE 7-16 Sect. 26.12

$GC_{pi} := .18$  +/- Internal Pressure Coefficients (ASCE 7-16 Table 26.13-1)

Velocity Pressure Exposure Coefficient (Table 26.10-1):

$z_g := 12200\text{ft}$   $\alpha := 7.0$  (per ASCE 7-16 Table 26.11-1 based on Exposure Category)  
 $z_g = 1200\text{ft}, \alpha = 7.0$  (Exp B),  $z_g = 900\text{ft}, \alpha = 9.5$  (Exp C),  $z_g = 700\text{ft}, \alpha = 11.5$  (Exp D)

$z_1 := 18\text{ft}$   $z_2 := 15\text{ft}$  Height from ground to level x ( $z_{\text{min}} = 15\text{ft}$ )

$$K_{z1} := 2.01 \left( \frac{z_1}{z_g} \right)^{\left( \frac{2}{\alpha} \right)} = 0.31 \quad K_{z2} := 2.01 \left( \frac{z_2}{z_g} \right)^{\left( \frac{2}{\alpha} \right)} = 0.3 \quad K_h := 2.01 \left( \frac{h}{z_g} \right)^{\left( \frac{2}{\alpha} \right)} = 0.31$$

External Pressure Coefficients w/ Roof Pitch = 1/12 (5 degrees) Front to Back & 1/12 (5 degrees) Side to Side  
 Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 55\text{ft}$   $B_{fb} := 55\text{ft}$   $\frac{L_{fb}}{B_{fb}} = 1$   $\frac{h}{L_{fb}} = 0.33$

Side to Side:

$L_{ss} := 55\text{ft}$   $B_{ss} := 55\text{ft}$   $\frac{L_{ss}}{B_{ss}} = 1$   $\frac{h}{L_{ss}} = 0.33$

$C_{pf1} := .8$  Windward Wall

$C_{ps1} := .8$  Windward Wall

$C_{pf2} := -0.18$  Windward Roof

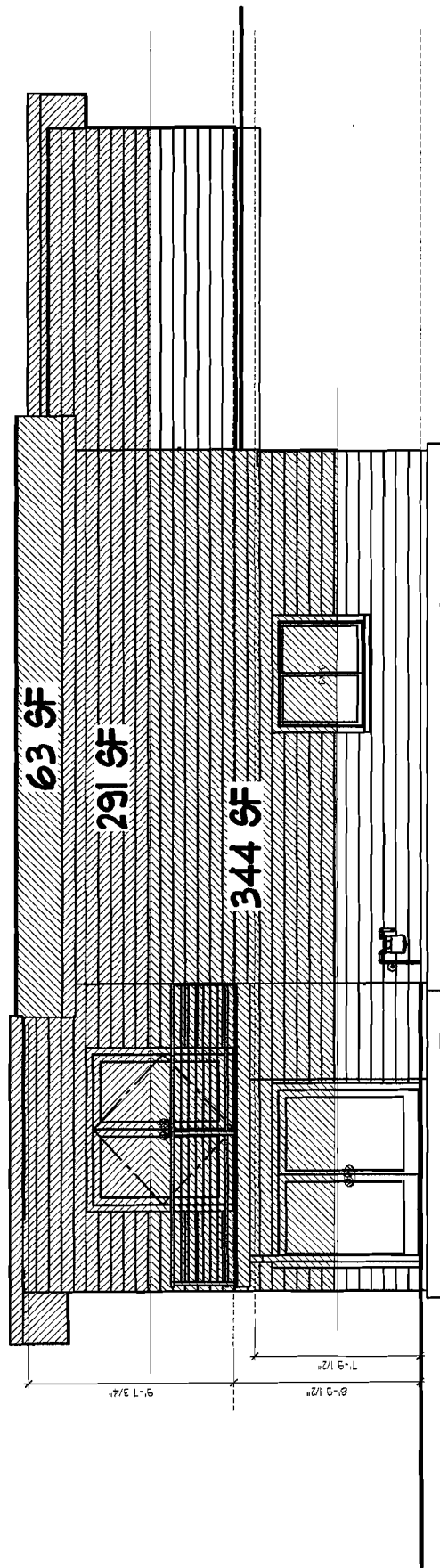
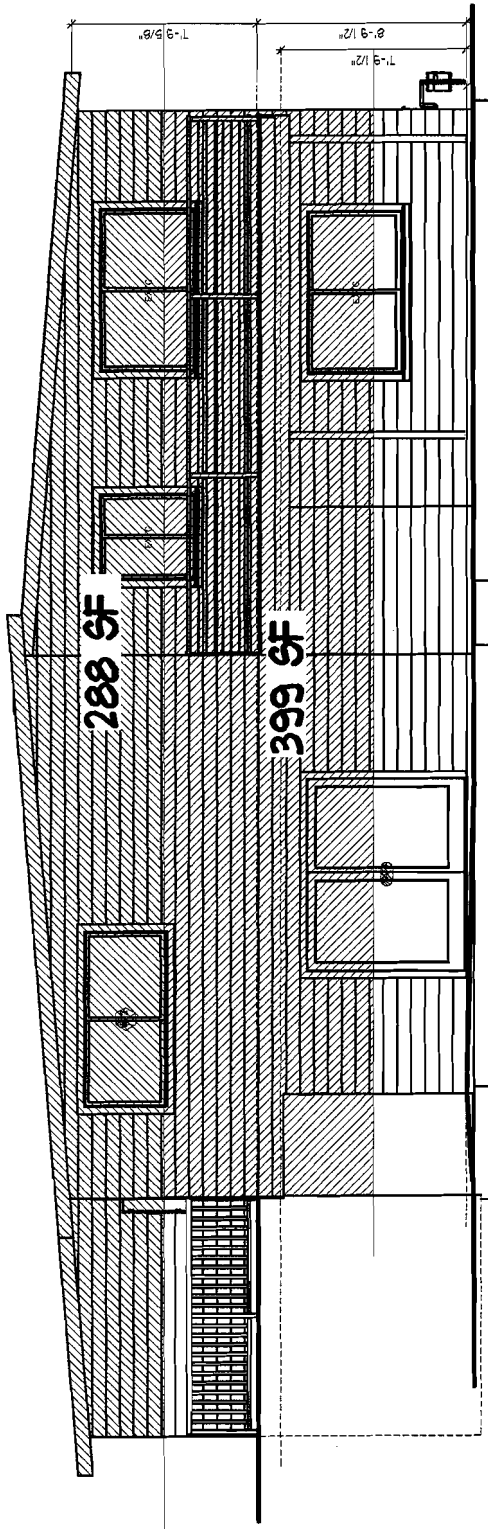
$C_{ps2} := -0.18$  Windward Roof

$C_{pf3} := -0.18$  Leeward Roof

$C_{ps3} := -0.18$  Leeward Roof

$C_{pf4} := -0.5$  Leeward Wall

$C_{ps4} := -0.5$  Leeward Wall



Velocity Pressure ( $q_z$ ) Evaluated at Height (z) (Equation 26.10-1)

$$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 6.79 \quad q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 6.45 \quad q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 6.79$$

Design Wind Pressures  $p = qGC_p - q_i(GC_{pi})$  (Equation 27.3-1) where  $q_i$  will conservatively be taken equal to  $q_h$

Windward Wall Both Directions  $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 4.62 \text{ ft}^{-2} \cdot \text{lb}$   $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 4.38 \text{ ft}^{-2} \cdot \text{lb}$

Windward Roof Front to Back  $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -1.04 \text{ ft}^{-2} \cdot \text{lb}$

Leeward Roof Front to Back  $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -1.04 \text{ ft}^{-2} \cdot \text{lb}$

Leeward Wall Front to Back  $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -2.89 \text{ ft}^{-2} \cdot \text{lb}$

Windward Roof Side to Side  $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = -1.04 \text{ ft}^{-2} \cdot \text{lb}$

Leeward Roof Side to Side  $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -1.04 \text{ ft}^{-2} \cdot \text{lb}$

Leeward Wall Side to Side  $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -2.89 \text{ ft}^{-2} \cdot \text{lb}$

The Internal Pressures on Windward and Leeward Walls & Roofs will offset each other for the lateral design of the overall building and will therefore be ignored for this application.

Check net pressure not less than 16psf at walls & 8psf at roof over projected vertical plane per ASCE 7-16 Sec. 27.1-5:

$$p_{wr1} - p_{lr1} = 0 \text{ ft}^{-2} \cdot \text{lb} \quad p_{ww1} - p_{lw1} = 7.5 \text{ ft}^{-2} \cdot \text{lb} \quad p_{ww2} - p_{lw1} = 7.27 \text{ ft}^{-2} \cdot \text{lb}$$

$$p_{wr2} - p_{lr2} = 0 \text{ ft}^{-2} \cdot \text{lb} \quad p_{ww1} - p_{lw2} = 7.5 \text{ ft}^{-2} \cdot \text{lb} \quad p_{ww2} - p_{lw2} = 7.27 \text{ ft}^{-2} \cdot \text{lb}$$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (8\text{psf})0\text{ft}^2 + (16\text{psf}) \cdot 288 \cdot \text{ft}^2 = 4608 \text{ lb}$$

Wind Pressure at Main Floor (Front to Back):

$$V_{2W} := (8\text{psf})0\text{ft}^2 + (16\text{psf}) \cdot 399 \cdot \text{ft}^2 = 6384 \text{ lb}$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (8\text{psf}) \cdot 63\text{ft}^2 + (16\text{psf}) \cdot 291\text{ft}^2 = 5160 \text{ lb}$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (8\text{psf}) \cdot 0\text{ft}^2 + (16\text{psf}) \cdot 344\text{ft}^2 = 5504 \text{ lb}$$



**WALL AA:**

Story Shear due to Wind:  $V_{3W} = 5160 \text{ lb}$  Story Shear due to Seismic:  $F_1 = 8859.16 \text{ lb}$

Bldg Width in direction of Load:  $L_t := 54.5 \text{ ft}$  Distance between shear walls:  $L_1 := 14.5 \text{ ft}$

Shear Wall Length:  $L_{aa} := \left[ 3.83 \left[ 1.25 - 0.125 \left( \frac{8}{3.83} \right) \right] + 11.58 \right] \text{ ft} = 15.37 \text{ ft}$

Percent full height sheathing:  $\%_{\text{sheath}} := \left( \frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \right) \cdot 100 \quad \% = 100$  Max Opening Height = 0ft-0in, Therefore  $C_o := 1.00$  per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $v_{aa} := \frac{0.6 V_{3W} \cdot L_1}{L_t \cdot 2} \cdot \frac{1}{L_{aa}}$

Seismic Force:  $\rho := 1.0 \quad E_{aa} := \frac{\rho \cdot 0.7 F_1 \cdot L_1}{L_t \cdot 2} \cdot \frac{1}{L_{aa}}$

$v_{aa} = 26.8 \text{ ft}^{-1} \cdot \text{lb}$

$\frac{v_{aa}}{C_o} = 26.8 \text{ ft}^{-1} \cdot \text{lb}$

$E_{aa} = 53.68 \text{ ft}^{-1} \cdot \text{lb}$

$\frac{E_{aa}}{C_o} = 53.68 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**

Wind Capacity = 339 plf

Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_{aa} := 3.83 \cdot \text{ft}$  Plate Height:  $P_t := 8 \cdot \text{ft}$

$W_{aa} := (15 \cdot \text{psf}) \cdot 3 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLR_{aa} := \frac{W_{aa} \cdot L_{aa}}{2} \quad DLR_{aa} = 239.37 \text{ lb}$

Chord Force:

$CF_{aa_w} := \frac{v_{aa} \cdot L_{aa} \cdot P_t}{C_o \cdot L_{aa}} \quad CF_{aa_w} = 214.4 \text{ lb}$

$CF_{aa_s} := \frac{E_{aa} \cdot L_{aa} \cdot P_t}{C_o \cdot L_{aa}} \quad CF_{aa_s} = 429.46 \text{ lb}$

Holddown Force:

$HDF_{aa_w} := CF_{aa_w} - 0.6 \cdot DLR_{aa} = 70.78 \text{ lb}$

$HDF_{aa_s} := CF_{aa_s} - (0.6 - 0.14 S_{DS}) DLR_{aa} = 317.33 \text{ lb}$

No Holddown Required

Base Plate Nail Spacing (2018 NDS Table 12N)

16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot \text{lb} \quad C_D := 1.6$

$B_p := \frac{(Z_N \cdot C_D \cdot C_o)}{v_{aa}} = 6.09 \text{ ft} \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 3.04 \text{ ft}$

16d @ 16" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot \text{lb} \quad C_{DW} := 1.6 \quad Z_B := A_s \cdot C_D \quad Z_B = 1376 \text{ lb}$

$A_s := \frac{(Z_B \cdot C_o)}{v_{aa}} = 51.34 \text{ ft} \quad \frac{(Z_B \cdot C_o)}{E_{aa}} = 25.63 \text{ ft}$

5/8" A.B. @ 72" o.c.

**WALL BB:**

Story Shear due to Wind:  $V_{3W} = 5160 \text{ lb}$  Story Shear due to Seismic:  $F_1 = 8859.16 \text{ lb}$

Bldg Width in direction of Load:  $L_{\text{wall}} := 54.5 \text{ ft}$  Distance between shear walls:  $L_{1W} := 14.5 \text{ ft}$   $L_2 := 25 \text{ ft}$

Shear Wall Length:  $L_{bb} := \left[ 3.17 \left[ 1.25 - 0.125 \left( \frac{9.5}{3.17} \right) \right] + 5.08 + 4 \right] \text{ ft} = 11.86 \text{ ft}$

Percent full height sheathing:  $\%_{\text{sheath}} := \left( \frac{10 \text{ ft}}{10 \text{ ft}} \right) \cdot 100$   $\% = 100$  Max Opening Height = 4ft-0in, Therefore  $C_{\text{max}} := 1.00$  per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $v_{bb} := \frac{0.6V_{3W} \cdot L_1 + L_2}{L_t \cdot 2}$

Seismic Force:  $\rho_{\text{max}} := 1.0$   $E_{bb} := \frac{0.7F_1 \cdot L_1 + L_2}{L_t \cdot 2}$

$v_{bb} = 94.64 \text{ ft}^{-1} \cdot \text{lb}$

$\frac{v_{bb}}{C_o} = 94.64 \text{ ft}^{-1} \cdot \text{lb}$

$E_{bb} = 189.57 \text{ ft}^{-1} \cdot \text{lb}$

$\frac{E_{bb}}{C_o} = 189.57 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**  
Wind Capacity = 339 plf  
Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_{bb} := 3.17 \text{ ft}$  Plate Height:  $P_t := 9.5 \text{ ft}$

$W_{bb} := (15 \text{ psf}) \cdot 2 \text{ ft} + (10 \text{ psf}) \cdot P_t + (10 \text{ psf}) \cdot 0 \text{ ft}$

$DLR_{bb} := \frac{W_{bb} \cdot L_{bb}}{2}$   $DLR_{bb} = 198.13 \text{ lb}$

Chord Force:

$CF_{bb_w} := \frac{v_{bb} \cdot L_{bb} \cdot P_t}{C_o \cdot L_{bb}}$   $CF_{bb_w} = 899.07 \text{ lb}$

$CF_{bb_s} := \frac{E_{bb} \cdot L_{bb} \cdot P_t}{C_o \cdot L_{bb}}$   $CF_{bb_s} = 1800.87 \text{ lb}$

Holdown Force:

$HDF_{bb_w} := CF_{bb_w} - 0.6 \cdot DLR_{bb} = 780.2 \text{ lb}$

$HDF_{bb_s} := CF_{bb_s} - (0.6 - 0.14S_{DS}) \cdot DLR_{bb} = 1708.07 \text{ lb}$

Simpson MST37 or MSTC40 strap with DTT2Z holdown at 1/2" dia all-thread anchor epoxied at foundation

Base Plate Nail Spacing (2018 NDS Table 12N)

16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_{\text{max}} := 102 \cdot \text{lb}$   $C_{\text{DW}} := 1.6$   
 $B_{\text{max}} := \frac{(C_D \cdot Z_N \cdot C_o)}{v_{bb}} = 1.72 \text{ ft}$   $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.86 \text{ ft}$

16d @ 8" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_{\text{max}} := 860 \cdot \text{lb}$   $C_{\text{DW}} := 1.6$   $Z_B := A_s \cdot C_D$   $Z_B = 1376 \text{ lb}$   
 $A_{\text{max}} := \frac{(Z_B \cdot C_o)}{v_{bb}} = 14.54 \text{ ft}$   $\frac{(Z_B \cdot C_o)}{E_{bb}} = 7.26 \text{ ft}$

5/8" A.B. @ 72" o.c.

**WALL CC:**

Story Shear due to Wind:  $V_{IW} = 4608 \text{ lb}$  Story Shear due to Seismic:  $F_1 = 8859.16 \text{ lb}$

Bldg Width in direction of Load:  $L_{W1} := 55.25 \text{ ft}$  Distance between shear walls:  $L_{W1} := 10 \text{ ft}$   $L_{W2} := 10 \text{ ft}$

Shear Wall Length:  $L_{cc} := (13.17 + 5.25) \text{ ft} = 18.42 \text{ ft}$

Percent full height sheathing:  $\%_{\text{sheath}} := \left( \frac{10 \text{ ft}}{10 \text{ ft}} \right) \cdot 100$   $\% = 100$  Max Opening Height = 0ft-0in, Therefore  $C_{\text{max}} := 1.00$   
per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $v_{cc} := \frac{0.6V_{IW} \cdot \frac{L_1 + L_2}{L_t} \cdot \frac{1}{2}}{L_{cc}}$  Seismic Force:  $\rho_s := 1.0$   $E_{cc} := \frac{\rho_s \cdot \frac{0.7F_1 \cdot \frac{L_1 + L_2}{L_t} \cdot \frac{1}{2}}{L_{cc}}}$

$v_{cc} = 27.17 \text{ ft}^{-1} \cdot \text{lb}$   $\frac{v_{cc}}{C_o} = 27.17 \text{ ft}^{-1} \cdot \text{lb}$   $E_{cc} = 60.94 \text{ ft}^{-1} \cdot \text{lb}$   $\frac{E_{cc}}{C_o} = 60.94 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**  
Wind Capacity = 339 plf  
Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_{cc} := 5.25 \text{ ft}$  Plate Height:  $P_t := 7.75 \text{ ft}$

$W_{cc} := (15 \cdot \text{psf}) \cdot 6.5 \text{ ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 0 \text{ ft}$   $DLR_{cc} := \frac{W_{cc} \cdot L_{cc}}{2}$   $DLR_{cc} = 459.38 \text{ lb}$

Chord Force:

$CF_{ccw} := \frac{v_{cc} \cdot L_{cc} \cdot P_t}{C_o \cdot L_{cc}}$   $CF_{ccw} = 210.54 \text{ lb}$   $CF_{ccs} := \frac{E_{cc} \cdot L_{cc} \cdot P_t}{C_o \cdot L_{cc}}$   $CF_{ccs} = 472.25 \text{ lb}$

Holdown Force:

$HDF_{ccw} := CF_{ccw} - 0.6DLR_{cc} = -65.08 \text{ lb}$   $HDF_{ccs} := CF_{ccs} - (0.6 - 0.14S_{DS}) \cdot DLR_{cc} = 257.08 \text{ lb}$

No Holdown Required

Base Plate Nail Spacing (2018 NDS Table 12N)  
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_{\text{max}} := 102 \cdot \text{lb}$   $C_{\text{ND}} := 1.6$   
 $B_{\text{max}} := \frac{(C_D \cdot Z_N \cdot C_o)}{v_{cc}} = 6.01 \text{ ft}$   $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 2.68 \text{ ft}$

16d @ 16" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)  
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot \text{lb}$   $C_{\text{ND}} := 1.6$   $Z_B := A_s \cdot C_D$   $Z_B = 1376 \text{ lb}$   
 $A_{s1} := \frac{(Z_B \cdot C_o)}{v_{cc}} = 50.65 \text{ ft}$   $\frac{(Z_B \cdot C_o)}{E_{cc}} = 22.58 \text{ ft}$

5/8" A.B. @ 72" o.c.

**WALL DD:**

Story Shear due to Wind:  $V_{1W} = 4608 \text{ lb}$       Story Shear due to Seismic:  $F_1 = 8859.16 \text{ lb}$   
 Bldg Width in direction of Load:  $L_{\text{W}} := 55.25 \text{ ft}$       Distance between shear walls:  $L_{1W} := 12.5 \text{ ft}$        $L_{2W} := 23.25 \text{ ft}$   
 Shear Wall Length:  $L_{\text{dd}} := \left[ 4 \left[ 1.25 + 0.125 \left( \frac{9.66}{4} \right) \right] + 10.58 + 9.92 \right] \text{ ft} = 26.71 \text{ ft}$

Percent full height sheathing:  $\%_{\text{W}} := \left( \frac{10 \text{ ft}}{10 \text{ ft}} \right) \cdot 100$        $\% = 100$       Max Opening Height = 0ft-0in, Therefore  $C_{\text{max}} := 1.00$   
 per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $v_{\text{dd}} := \frac{0.6 V_{1W} \cdot \frac{L_1 + L_2}{L_t \cdot 2}}{L_{\text{dd}}}$       Seismic Force:  $\rho_{\text{W}} := 1.0$        $E_{\text{dd}} := \frac{\rho \cdot \frac{0.7 F_1 \cdot (L_1 + L_2)}{L_t \cdot 2}}{L_{\text{dd}}}$

$v_{\text{dd}} = 33.49 \text{ ft}^{-1} \cdot \text{lb}$        $\frac{v_{\text{dd}}}{C_o} = 33.49 \text{ ft}^{-1} \cdot \text{lb}$        $E_{\text{dd}} = 75.12 \text{ ft}^{-1} \cdot \text{lb}$        $\frac{E_{\text{dd}}}{C_o} = 75.12 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**  
 Wind Capacity = 339 plf  
 Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_{\text{dd}} := 4 \text{ ft}$       Plate Height:  $P_t := 9.66 \text{ ft}$   
 $W_{\text{dd}} := (15 \text{ psf}) \cdot 7.5 \text{ ft} + (10 \text{ psf}) \cdot P_t + (10 \text{ psf}) \cdot 0 \text{ ft}$        $\text{DLR}_{\text{dd}} := \frac{W_{\text{dd}} \cdot L_{\text{dd}}}{2}$        $\text{DLR}_{\text{dd}} = 418.2 \text{ lb}$

Chord Force:  
 $\text{CF}_{\text{dd}_w} := \frac{v_{\text{dd}} \cdot L_{\text{dd}} \cdot P_t}{C_o \cdot L_{\text{dd}}}$        $\text{CF}_{\text{dd}_w} = 323.54 \text{ lb}$        $\text{CF}_{\text{dd}_s} := \frac{E_{\text{dd}} \cdot L_{\text{dd}} \cdot P_t}{C_o \cdot L_{\text{dd}}}$        $\text{CF}_{\text{dd}_s} = 725.69 \text{ lb}$

Holdown Force:  
 $\text{HDF}_{\text{dd}_w} := \text{CF}_{\text{dd}_w} - 0.6 \text{DLR}_{\text{dd}} = 72.62 \text{ lb}$        $\text{HDF}_{\text{dd}_s} := \text{CF}_{\text{dd}_s} - (0.6 - 0.14 S_{\text{DS}}) \text{DLR}_{\text{dd}} = 529.8 \text{ lb}$

No Holdown Required

**Base Plate Nail Spacing (2018 NDS Table 12N)**  
 16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_{\text{N}} := 102 \text{ lb}$        $C_{\text{D}} := 1.6$   
 $B_{\text{N}} := \frac{(C_{\text{D}} \cdot Z_{\text{N}} \cdot C_o)}{v_{\text{dd}}} = 4.87 \text{ ft}$        $\frac{(C_{\text{D}} \cdot Z_{\text{N}} \cdot C_o)}{E_{\text{dd}}} = 2.17 \text{ ft}$

16d @ 16" o.c.

**Anchor Bolt Spacing (2018 NDS Table 12E)**  
 5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_{\text{S}} := 860 \text{ lb}$        $C_{\text{D}} := 1.6$        $Z_{\text{B}} := A_{\text{S}} \cdot C_{\text{D}}$        $Z_{\text{B}} = 1376 \text{ lb}$   
 $A_{\text{S}} := \frac{(Z_{\text{B}} \cdot C_o)}{v_{\text{dd}}} = 41.08 \text{ ft}$        $\frac{(Z_{\text{B}} \cdot C_o)}{E_{\text{dd}}} = 18.32 \text{ ft}$

5/8" A.B. @ 72" o.c.

**WALL EE:**

Story Shear due to Wind:  $V_{1W} = 4608 \text{ lb}$

Story Shear due to Seismic:  $F_1 = 8859.16 \text{ lb}$

Bldg Width in direction of Load:  $L_{\text{tot}} := 55.25 \text{ ft}$

Distance between shear walls:  $L_1 := 10 \text{ ft}$   $L_2 := 12.5 \text{ ft}$

Shear Wall Length:

$Lee := (21.25) \text{ ft} = 21.25 \text{ ft}$

Percent full height sheathing:  $\%_{\text{sheath}} := \left( \frac{10 \text{ ft}}{10 \text{ ft}} \right) \cdot 100$

$\% = 100$

Max Opening Height = 0ft-0in, Therefore  $C_{\text{max}} := 1.00$   
per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $vee := \frac{0.6V_{1W} \cdot \frac{L_1 + L_2}{2}}{Lee}$

Seismic Force:  $\rho_{\text{max}} := 1.0$

$E_{ee} := \frac{\rho \cdot \frac{0.7F_1 \cdot \frac{L_1 + L_2}{2}}{L_t}}{Lee}$

$vee = 26.49 \text{ ft}^{-1} \cdot \text{lb}$   $\frac{vee}{C_0} = 26.49 \text{ ft}^{-1} \cdot \text{lb}$

$E_{ee} = 59.42 \text{ ft}^{-1} \cdot \text{lb}$   $\frac{E_{ee}}{C_0} = 59.42 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**

Wind Capacity = 339 plf

Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_{ee} := 21.25 \text{ ft}$  Plate Height:  $P_t := 8.25 \text{ ft}$

$W_{ee} := (15 \cdot \text{psf}) \cdot 14 \text{ ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 0 \text{ ft}$

$DL_{Ree} := \frac{W_{ee} \cdot L_{ee}}{2}$   $DL_{Ree} = 3107.81 \text{ lb}$

Chord Force:

$CF_{ee_w} := \frac{vee \cdot L_{ee} \cdot P_t}{C_0 \cdot L_{ee}}$   $CF_{ee_w} = 218.56 \text{ lb}$

$CF_{ee_s} := \frac{E_{ee} \cdot L_{ee} \cdot P_t}{C_0 \cdot L_{ee}}$   $CF_{ee_s} = 490.24 \text{ lb}$

Holdown Force:

$HDF_{ee_w} := CF_{ee_w} - 0.6 \cdot DL_{Ree} = -1646.12 \text{ lb}$

$HDF_{ee_s} := CF_{ee_s} - (0.6 - 0.14S_{DS}) \cdot DL_{Ree} = -965.46 \text{ lb}$

No Holdown Required

Base Plate Nail Spacing (2018 NDS Table 12N)

16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_{N} := 102 \cdot \text{lb}$   $C_{\text{DN}} := 1.6$   
 $B_{\text{max}} := \frac{(Z_N \cdot C_D \cdot C_o)}{vee} = 6.16 \text{ ft}$   $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ee}} = 2.75 \text{ ft}$

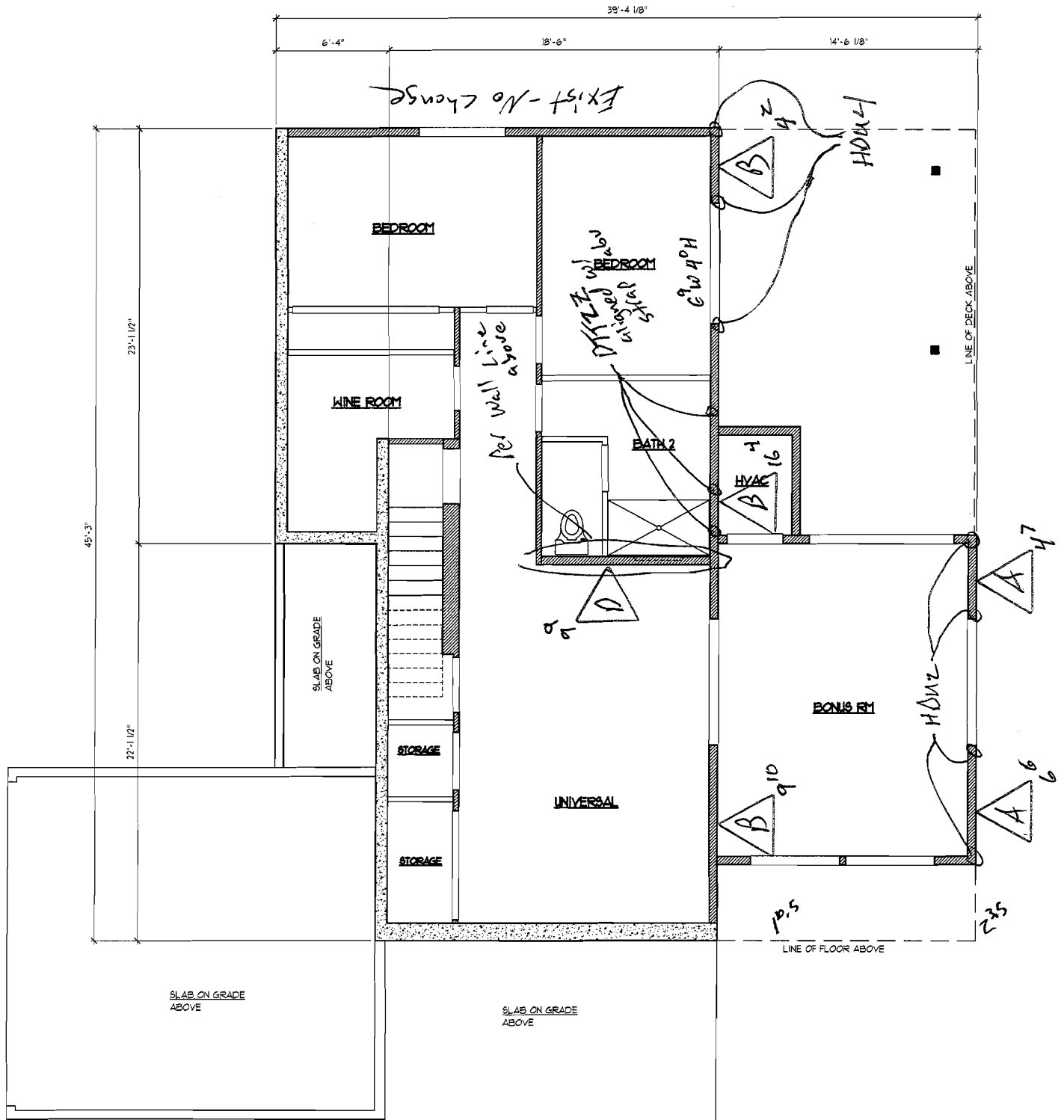
16d @ 16" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_{\text{max}} := 860 \cdot \text{lb}$   $C_{\text{DN}} := 1.6$   $Z_{\text{BA}} := A_s \cdot C_D$   $Z_B = 1376 \text{ lb}$   
 $A_{\text{Smax}} := \frac{(Z_B \cdot C_o)}{vee} = 51.94 \text{ ft}$   $\frac{(Z_B \cdot C_o)}{E_{ee}} = 23.16 \text{ ft}$

5/8" A.B. @ 72" o.c.



Majority of lateral load transferred to foundation by Main floor diaphragm. Check daylight wall affected by addition

**LOWER FLOOR PLAN**



**WALL A:**

Story Shear due to Wind:  $V_{4W} = 5504 \text{ lb}$  Story Shear due to Seismic:  $F_2 = 3402.41 \text{ lb}$

Bldg Width in direction of Load:  $L_{\text{M}} := 33\text{-ft}$  Distance between shear walls:  $L_{\text{W}} := 14.5\text{-ft}$

Shear Wall Length:  $L_a := (6.5 + 4.58)\text{ft} = 11.08 \text{ ft}$

Percent full height sheathing:  $\% := \left(\frac{10\text{-ft}}{10\text{-ft}}\right) \cdot 100 \quad \% = 100$  Max Opening Height = 0ft-0in, Therefore  $C_{\text{M}} := 1.00$   
per AF&PA SDPWS Table 4.3.3.5

Wind Force:  $v_a := \frac{v_{aa} \cdot L_{aa} + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{L_a}$  Seismic Force:  $E_a := \frac{E_{aa} \cdot L_{aa} + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{L_a}$

$v_a = 102.65 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{v_a}{C_o} = 102.65 \text{ ft}^{-1} \cdot \text{lb} \quad E_a = 121.68 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_a}{C_o} = 121.68 \text{ ft}^{-1} \cdot \text{lb}$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**  
Wind Capacity = 339 plf  
Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_a := 4.58\text{-ft}$  Plate Height:  $P_t := 7.75\text{-ft}$

$W_a := (15\text{-psf}) \cdot 0\text{-ft} + (10\text{-psf}) \cdot P_t + (10\text{psf}) \cdot 7\text{ft}$   $DLR_a := \frac{W_a \cdot L_a}{2} \quad DLR_a = 337.77 \text{ lb}$

Chord Force:

$CF_{a_w} := \frac{v_a \cdot L_a \cdot P_t}{C_o \cdot L_a} \quad CF_{a_w} = 795.55 \text{ lb} \quad CF_{a_s} := \frac{E_a \cdot L_a \cdot P_t}{C_o \cdot L_a} \quad CF_{a_s} = 943.02 \text{ lb}$   
 $CF_{a_w} + CF_{a_{a_w}} = 1009.95 \text{ lb} \quad CF_{a_s} + CF_{a_{a_s}} = 1372.47 \text{ lb}$

Holdown Force:

$HDF_{a_w} := CF_{a_w} - 0.6 \cdot DLR_a = 592.89 \text{ lb} \quad HDF_{a_s} := CF_{a_s} - (0.6 - 0.14S_{D_S}) \cdot DLR_a = 784.8 \text{ lb}$   
 $HDF_{a_w} + HDF_{a_{a_w}} = 663.66 \text{ lb} \quad HDF_{a_s} + HDF_{a_{a_s}} = 1102.13 \text{ lb}$

Simpson HDU2 w/ SSTB16 anchor

Base Plate Nail Spacing (2018 NDS Table 12N)

16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

Anchor Bolt Spacing (2018 NDS Table 12E)

5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$Z_{\text{M}} := 102 \cdot \text{lb} \quad C_{D_V} := 1.6$   
 $B_{\text{M}} := \frac{(C_D \cdot Z_N \cdot C_o)}{v_a} = 1.59 \text{ ft} \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 1.34 \text{ ft}$

$A_{\text{M}} := 860 \cdot \text{lb} \quad C_{D_V} := 1.6 \quad Z_{B_A} := A_s \cdot C_D \quad Z_B = 1376 \text{ lb}$   
 $A_{\text{S}} := \frac{(Z_B \cdot C_o)}{v_a} = 13.4 \text{ ft} \quad \frac{(Z_B \cdot C_o)}{E_a} = 11.31 \text{ ft}$

16d @ 16" o.c.

5/8" A.B. @ 72" o.c.

**WALL B:**

Story Shear due to Wind:  $V_{4W} = 5504 \text{ lb}$  Story Shear due to Seismic:  $F_2 = 3402.41 \text{ lb}$

Bldg Width in direction of Load:  $L_{tt} := 33\text{-ft}$  Distance between shear walls:  $L_{ww} := 14.5\text{-ft}$   $L_{2w} := 18.5\text{ft}$

Shear Wall Length:  $L_b := (9.83 + 16.33 + 4.17)\text{ft} = 30.33 \text{ ft}$

Percent full height sheathing:  $\frac{\rho_{\text{sheath}}}{\rho_{\text{max}}} := \left(\frac{10\text{-ft}}{10\text{-ft}}\right) \cdot 100 \quad \% = 100$  Max Opening Height = 0ft-0in, Therefore  $C_{\text{oh}} := 1.00$  per AF&PA SDPWS Table 4.3.3.5

$$\text{Wind Force: } v_b := \frac{v_{bb} \cdot L_{bb} + \left(\frac{0.6V_{4W} \cdot L_1 + L_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{L_b} \quad \text{Seismic Force: } \rho_{\text{max}} := 1.0 \quad E_b := \frac{E_{bb} \cdot L_{bb} + \left(\rho \cdot \frac{0.7F_2 \cdot L_1 + L_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{L_b}$$

$$v_b = 91.43 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{v_b}{C_o} = 91.43 \text{ ft}^{-1} \cdot \text{lb} \quad E_b = 113.36 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_b}{C_o} = 113.36 \text{ ft}^{-1} \cdot \text{lb}$$

**P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.**  
Wind Capacity = 339 plf  
Seismic Capacity = 242 plf

Dead Load Resisting Overturning:  $L_b := 4.17\text{-ft}$  Plate Height:  $P_t := 7.75\text{-ft}$

$$W_b := (15\text{-psf}) \cdot 0\text{-ft} + (10\text{-psf}) \cdot P_t + (10\text{psf}) \cdot 10\text{ft} \quad \text{DLR}_b := \frac{W_b \cdot L_b}{2} \quad \text{DLR}_b = 370.09 \text{ lb}$$

Chord Force:

$$\text{CF}_{b_w} := \frac{v_b \cdot L_b \cdot P_t}{C_o \cdot L_b} \quad \text{CF}_{b_w} = 708.6 \text{ lb} \quad \text{CF}_{b_s} := \frac{E_b \cdot L_b \cdot P_t}{C_o \cdot L_b} \quad \text{CF}_{b_s} = 878.52 \text{ lb}$$

$$\text{CF}_{b_w} + \text{CF}_{b_{ww}} = 1607.67 \text{ lb} \quad \text{CF}_{b_s} + \text{CF}_{b_{ss}} = 2679.4 \text{ lb}$$

Holdown Force:

$$\text{HDF}_{b_w} := \text{CF}_{b_w} - 0.6 \cdot \text{DLR}_b = 486.55 \text{ lb} \quad \text{HDF}_{b_s} := \text{CF}_{b_s} - (0.6 - 0.14S_{DS}) \cdot \text{DLR}_b = 705.17 \text{ lb}$$

No Holdown Required

$$\text{HDF}_{b_w} + \text{HDF}_{b_{ww}} = 1266.74 \text{ lb} \quad \text{HDF}_{b_s} + \text{HDF}_{b_{ss}} = 2413.25 \text{ lb}$$

Simpson HDU4 w/ 5/8" dia all-thread anchor epoxied at foundation

Base Plate Nail Spacing (2018 NDS Table 12N)  
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$$Z_{N} := 102 \cdot \text{lb} \quad C_{D'} := 1.6$$

$$B_{N} := \frac{(C_{D'} \cdot Z_N \cdot C_o)}{v_b} = 1.78 \text{ ft} \quad \frac{(C_{D'} \cdot Z_N \cdot C_o)}{E_b} = 1.44 \text{ ft}$$

16d @ 4" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)  
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$$A_s := 860 \cdot \text{lb} \quad C_{D'} := 1.6 \quad Z_B := A_s \cdot C_{D'} \quad Z_B = 1376 \text{ lb}$$

$$A_{s'} := \frac{(Z_B \cdot C_o)}{v_b} = 15.05 \text{ ft} \quad \frac{(Z_B \cdot C_o)}{E_b} = 12.14 \text{ ft}$$

5/8" A.B. @ 32" o.c.

Diaphragm Shear Check:

Assume 2x HF Roof Framing, 7/16" Sheathing w/ 8d (0.131" x 2.5") nails, 6" o.c Edge nailing

Unblocked Diaphragm Case 1 Wind Capacity = 300 plf & Seismic Capacity = 214 plf

Unblocked Diaphragm Case 2-6 Wind Capacity = 221 plf & Seismic Capacity = 158 plf

Wall Lines AA:

$$v_{aa} \cdot \frac{L_{aa}}{23\text{ft}} = 17.91 \text{ ft}^{-1} \cdot \text{lb} \quad E_{aa} \cdot \frac{L_{aa}}{23\text{ft}} = 35.87 \text{ ft}^{-1} \cdot \text{lb}$$

Wall Lines BB:

$$v_{bb} \cdot \frac{L_{bb}}{45\text{ft}} = 24.93 \text{ ft}^{-1} \cdot \text{lb} \quad E_{bb} \cdot \frac{L_{bb}}{45\text{ft}} = 49.94 \text{ ft}^{-1} \cdot \text{lb}$$

Wall Lines CC:

$$v_{cc} \cdot \frac{L_{cc}}{54\text{ft}} = 9.27 \text{ ft}^{-1} \cdot \text{lb} \quad E_{cc} \cdot \frac{L_{cc}}{54\text{ft}} = 20.79 \text{ ft}^{-1} \cdot \text{lb}$$

Wall Lines A:

$$\frac{v_a \cdot L_a - v_{aa} \cdot L_{aa}}{23\text{ft}} = 31.54 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_a \cdot L_a - E_{aa} \cdot L_{aa}}{23\text{ft}} = 22.75 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{v_a \cdot L_a}{23\text{ft}} = 49.45 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_a \cdot L_a}{23\text{ft}} = 58.62 \text{ ft}^{-1} \cdot \text{lb}$$

Wall Lines B:

$$\frac{v_b \cdot L_b - v_{bb} \cdot L_{bb}}{45\text{ft}} = 36.69 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_b \cdot L_b - E_{bb} \cdot L_{bb}}{45\text{ft}} = 26.46 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{v_b \cdot L_b}{45\text{ft}} = 61.63 \text{ ft}^{-1} \cdot \text{lb} \quad \frac{E_b \cdot L_b}{45\text{ft}} = 76.4 \text{ ft}^{-1} \cdot \text{lb}$$

Wall Lines DD:

$$v_{dd} \cdot \frac{L_{dd}}{39.5\text{ft}} = 22.65 \text{ ft}^{-1} \cdot \text{lb} \quad E_{dd} \cdot \frac{L_{dd}}{39.5\text{ft}} = 50.79 \text{ ft}^{-1} \cdot \text{lb}$$

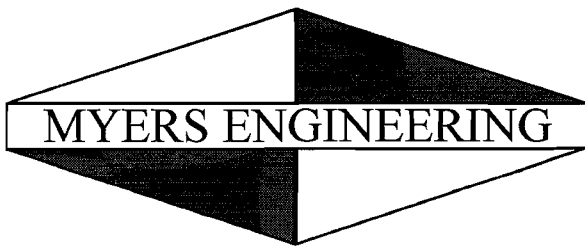
Wall Lines EE:

$$v_{ee} \cdot \frac{L_{ee}}{54\text{ft}} = 10.43 \text{ ft}^{-1} \cdot \text{lb} \quad E_{ee} \cdot \frac{L_{ee}}{54\text{ft}} = 23.38 \text{ ft}^{-1} \cdot \text{lb}$$



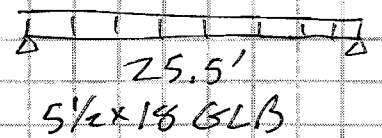




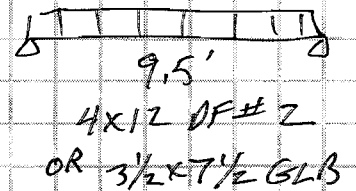


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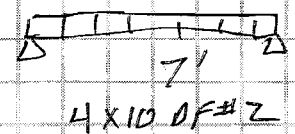
①  $w_D = 15 \text{ psf} \left(\frac{22'}{2}\right) = 165 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{22'}{2}\right) = 275 \text{ plf}$



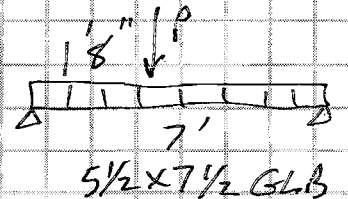
②  $w_D = 15 \text{ psf} \left(\frac{25'}{2}\right) = 187.5 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{25'}{2}\right) = 312.5 \text{ plf}$



③  $w_D = 187.5 \text{ plf}$   
 $w_S = 312.5 \text{ plf}$

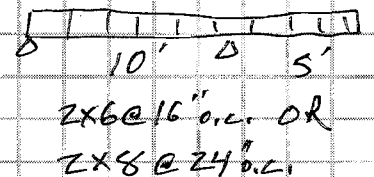


④  $w_D = 15 \text{ psf} \left(\frac{8'}{2}\right) = 60 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{8'}{2}\right) = 100 \text{ plf}$

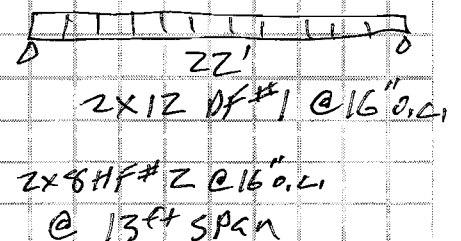


$P = 2105 \text{ \#DL} + 3510 \text{ \#SL from ①}$

⑤  $w_D = 15 \text{ psf}$   
 $w_S = 25 \text{ psf}$



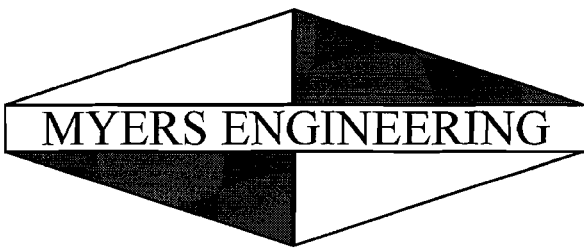
⑥  $w_D = 15 \text{ psf}$   
 $w_S = 25 \text{ psf}$



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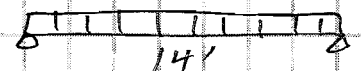
⑦  $w_D = 15 \text{ psf} \left(\frac{5'}{2}\right) = 37.5 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{5'}{2}\right) = 62.5 \text{ plf}$



(2) 2x12 DF#1

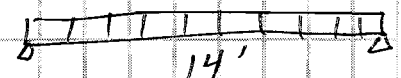
(2) 2x6 @ 13' span

⑧  $w_D = 15 \text{ psf}$   
 $w_L = 40 \text{ psf}$



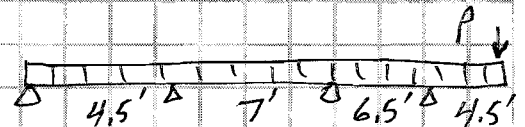
2x10 DF#2  
 @ 16" o.c.

⑨  $w_D = 15 \text{ psf} \left(\frac{25'}{2} + 1'\right) + 12 \text{ psf} (4') = 298.5 \text{ plf}$   
 $w_L = 40 \text{ psf} (1') = 40 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{25'}{2}\right) = 312.5 \text{ plf}$



5 1/2 x 10 1/2 GLB  
 Minimum

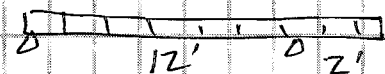
⑩  $w_D = 60 \text{ plf} + 120 \text{ plf} = 180 \text{ plf}$   
 $w_L = 40 \text{ psf} (14 1/2) = 280 \text{ plf}$   
 $w_S = 100 \text{ plf}$



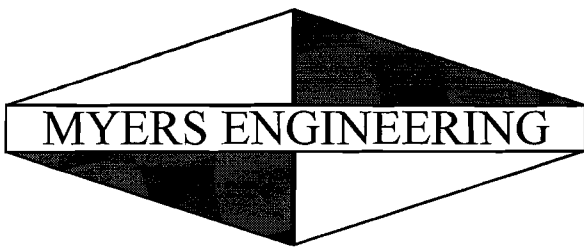
5 1/2 x 12 GLB  
 Minimum

$P = 2090^{\#} \text{ DL} + 2190^{\#} \text{ SL}$  from ⑨

⑪  $w_D = 10 \text{ psf}$   
 $w_L = 60 \text{ psf}$

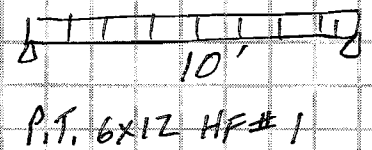


P.T. 2x10 @ 12" o.c.

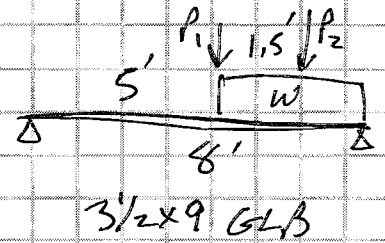


Myers Engineering LLC  
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(12)  $w_D = 10 \text{ psf} \left(\frac{16'}{2}\right) = 80 \text{ plf}$   
 $w_L = 60 \text{ psf} \left(\frac{16'}{2}\right) = 480 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{16'}{2}\right) = 200 \text{ plf}$



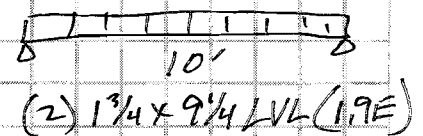
(13)  $w_D = 15 \text{ psf} \left(\frac{25'}{2} + 1'\right) + 12 \text{ psf} (12') = 481.5 \text{ plf}$   
 $w_L = 40 \text{ plf} + 60 \text{ plf} = 100 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{25'}{2} + 1'\right) = 337.5 \text{ plf}$



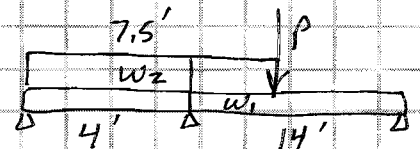
$P_1 = 660 \text{ DL} + 1090 \text{ SL}$  from (3)

$P_2 = 400 \text{ DL} + 2400 \text{ DL} + 1000 \text{ SL}$  from (12)

(14)  $w_D = 15 \text{ psf} \left(\frac{45'}{2}\right) = 337.5 \text{ plf}$   
 $w_S = 25 \text{ psf} \left(\frac{45'}{2}\right) = 562.5 \text{ plf}$



(15)  $w_{D1} = 15 \text{ psf} (1.33') = 20 \text{ plf}$   
 $w_{L1} = 40 \text{ psf} (1.33') = 53.3 \text{ plf}$



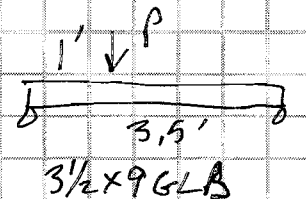
$w_{D2} = 15 \text{ psf} \left(\frac{22'}{2}\right) = 165 \text{ plf}$   
 $w_{S2} = 25 \text{ psf} \left(\frac{22'}{2}\right) = 275 \text{ plf}$

(4)  $1 \frac{3}{4} \times 9 \frac{1}{4}$  LVL @ 14'

$P = 2100 \text{ DL} + 3510 \text{ SL}$  from (1)

(1)  $1 \frac{3}{4} \times 9 \frac{1}{4}$  LVL @ 4'

(16)  $w_D = 15 \text{ psf} \left(\frac{18'}{2}\right) = 135 \text{ plf}$   
 $w_L = 40 \text{ psf} \left(\frac{18'}{2}\right) = 360 \text{ plf}$



$P = 2590 \text{ DL} + 4020 \text{ SL}$  from (15)

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DATE 9-7-24  
 BY ML

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Description :**

**Wood Beam Design :** 1. Roof Beam Option

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 5.5x18, GLB, Fully Braced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension 2,400.0 psi Fc - Prll 1,650.0 psi Fv 265.0 psi Ebend- xx 1,800.0 ksi Density 31.210 pcf  
 Fb - Compr 1,850.0 psi Fc - Perp 650.0 psi Ft 1,100.0 psi Eminbend - xx 950.0 ksi

Applied Loads

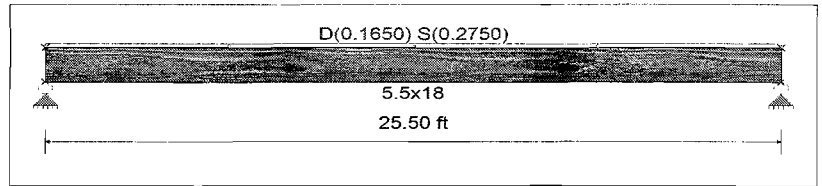
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 11.0 ft

Design Summary

Max fb/Fb Ratio = 0.560 : 1  
 fb : Actual : 1,445.00 psi at 12.750 ft in Span # 1  
 Fb : Allowable : 2,581.08 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.247 : 1  
 fv : Actual : 75.37 psi at 24.055 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 2.10 3.51  
 Right Support 2.10 3.51



Max Deflections			
Transient Downward	0.547 in	Total Downward	0.875 in
Ratio	559	Ratio	349
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design :** 2. Header at Patio Door

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 4x12, Sawn, Fully Unbraced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DouglasFir-Larch

Wood Grade : No.2

Fb - Tension 900.0 psi Fc - Prll 1,350.0 psi Fv 180.0 psi Ebend- xx 1,600.0 ksi Density 31.210 pcf  
 Fb - Compr 900.0 psi Fc - Perp 625.0 psi Ft 575.0 psi Eminbend - xx 580.0 ksi

Applied Loads

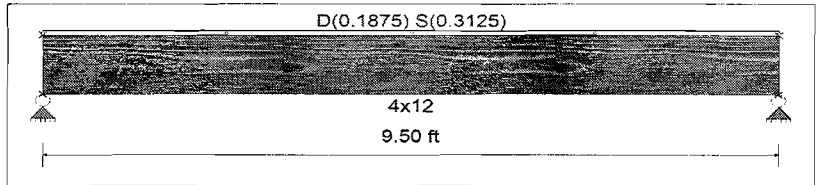
Unif Load: D = 0.1875, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.824 : 1  
 fb : Actual : 916.83 psi at 4.750 ft in Span # 1  
 Fb : Allowable : 1,112.07 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.353 : 1  
 fv : Actual : 72.98 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 207.00 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 0.89 1.48  
 Right Support 0.89 1.48



Max Deflections			
Transient Downward	0.087 in	Total Downward	0.139 in
Ratio	1315	Ratio	822
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 2. Header at Patio Door**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **3.5x7.5, GLB, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension 2,400.0 psi Fc - Prll 1,650.0 psi Fv 265.0 psi Ebend- xx 1,800.0 ksi Density 31.210 pcf  
 Fb - Compr 1,850.0 psi Fc - Perp 650.0 psi Ft 1,100.0 psi Eminbend - xx 950.0 ksi

Applied Loads

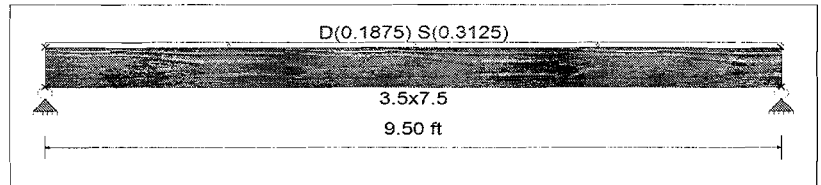
Unif Load: D = 0.1875, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.759 : 1**  
 fb : Actual : 2,062.86 psi at 4.750 ft in Span # 1  
 Fb : Allowable : 2,718.01 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.389 : 1**  
 fv : Actual : 118.52 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{E}$   
 Left Support 0.89 1.48  
 Right Support 0.89 1.48



Max Deflections

Transient Downward 0.260 in Total Downward 0.416 in  
 Ratio 438 Ratio 274  
 LC: S Only LC: +D+S  
 Transient Upward 0.000 in Total Upward 0.000 in  
 Ratio 9999 Ratio 9999  
 LC: LC:

**Wood Beam Design : 3. Header at Deck Door**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **4x10, Sawn, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch

Wood Grade : No.2

Fb - Tension 900.0 psi Fc - Prll 1,350.0 psi Fv 180.0 psi Ebend- xx 1,600.0 ksi Density 31.210 pcf  
 Fb - Compr 900.0 psi Fc - Perp 625.0 psi Ft 575.0 psi Eminbend - xx 580.0 ksi

Applied Loads

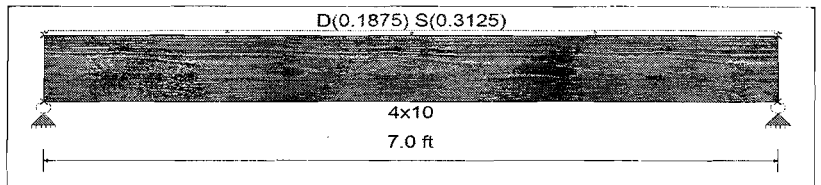
Unif Load: D = 0.1875, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.601 : 1**  
 fb : Actual : 736.30 psi at 3.500 ft in Span # 1  
 Fb : Allowable : 1,224.90 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.306 : 1**  
 fv : Actual : 63.24 psi at 6.230 ft in Span # 1  
 Fv : Allowable : 207.00 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{E}$   
 Left Support 0.66 1.09  
 Right Support 0.66 1.09



Max Deflections

Transient Downward 0.046 in Total Downward 0.074 in  
 Ratio 1827 Ratio 1142  
 LC: S Only LC: +D+S  
 Transient Upward 0.000 in Total Upward 0.000 in  
 Ratio 9999 Ratio 9999  
 LC: LC:

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 4. Header at Kitchen Window**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **5.5x7.5, GLB, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension 2,400.0 psi Fc - Prll 1,650.0 psi Fv 265.0 psi Ebend- xx 1,800.0 ksi Density 31.210 pcf  
 Fb - Compr 1,850.0 psi Fc - Perp 650.0 psi Ft 1,100.0 psi Eminbend - xx 950.0 ksi

Applied Loads

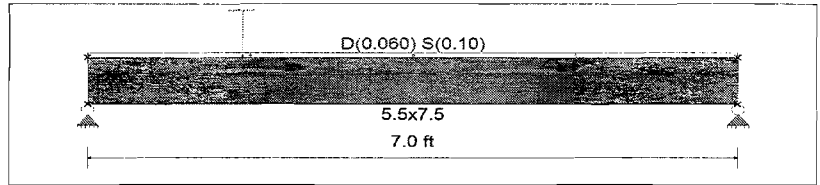
Unif Load: D = 0.060, S = 0.10 k/ft, Trib= 1.0 ft  
 1Point: D = 2.105, S = 3.510 k @ 1.667 ft

Design Summary

Max fb/Fb Ratio = **0.663 : 1**  
 fb : Actual : 1,821.97 psi at 1.680 ft in Span # 1  
 Fb : Allowable : 2,749.13 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.566 : 1**  
 fv : Actual : 172.39 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 1.81 0.71 3.02  
 Right Support 0.71 1.19



Max Deflections

Transient Downward 0.100 in Total Downward 0.159 in  
 Ratio 843 Ratio 527  
 LC: S Only LC: +D+S  
 Transient Upward 0.000 in Total Upward 0.000 in  
 Ratio 9999 Ratio 9999  
 LC: LC:

**Wood Beam Design : 5. Entry sistered overhangs**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2x6, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Hem-Fir

Wood Grade : No.2

Fb - Tension 850.0 psi Fc - Prll 1,300.0 psi Fv 150.0 psi Ebend- xx 1,300.0 ksi Density 26.840 pcf  
 Fb - Compr 850.0 psi Fc - Perp 405.0 psi Ft 525.0 psi Eminbend - xx 470.0 ksi

Applied Loads

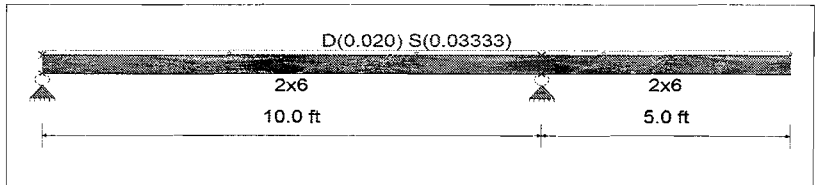
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 1.333 ft

Design Summary

Max fb/Fb Ratio = **0.724 : 1**  
 fb : Actual : 1,057.59 psi at 10.000 ft in Span # 1  
 Fb : Allowable : 1,461.36 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.326 : 1**  
 fv : Actual : 56.23 psi at 9.550 ft in Span # 1  
 Fv : Allowable : 172.50 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 0.07 0.22 0.12  
 Right Support 0.22 0.37



Max Deflections

Transient Downward 0.166 in Total Downward 0.265 in  
 Ratio 722 Ratio 452  
 LC: S Only LC: +D+S  
 Transient Upward 0.000 in Total Upward 0.000 in  
 Ratio 9999 Ratio 9999  
 LC: LC:

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KWV-06015659, Build:20.24.08.01

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**Wood Beam Design : 5. Entry sistered overhangs**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2x8, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Hem-Fir Wood Grade : No.2  
 Fb - Tension 850.0 psi Fc - Prll 1,300.0 psi Fv 150.0 psi Ebend- xx 1,300.0 ksi Density 26.840 pcf  
 Fb - Compr 850.0 psi Fc - Perp 405.0 psi Ft 525.0 psi Eminbend - xx 470.0 ksi

Applied Loads

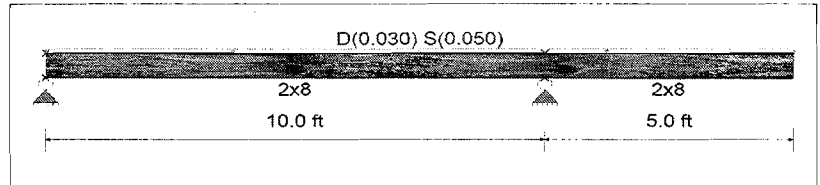
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.677 : 1**  
 fb : Actual : 913.20 psi at 10.000 ft in Span # 1  
 Fb : Allowable : 1,348.95 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.400 : 1**  
 fv : Actual : 68.97 psi at 10.000 ft in Span # 1  
 Fv : Allowable : 172.50 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{H}$   
 Left Support 0.11 0.19 0.56  
 Right Support 0.34 0.56



Max Deflections

Transient Downward	0.109 in	Total Downward	0.174 in
Ratio	1104	Ratio	690
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design : 6. New Roof Joists**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2x12, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch Wood Grade : No.1  
 Fb - Tension 1,000.0 psi Fc - Prll 1,500.0 psi Fv 180.0 psi Ebend- xx 1,700.0 ksi Density 31.210 pcf  
 Fb - Compr 1,000.0 psi Fc - Perp 625.0 psi Ft 675.0 psi Eminbend - xx 620.0 ksi

Applied Loads

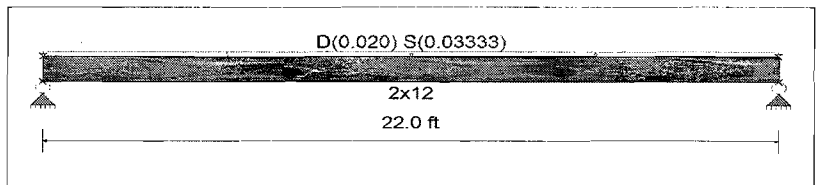
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 1.333 ft

Design Summary

Max fb/Fb Ratio = **0.925 : 1**  
 fb : Actual : 1,223.44 psi at 11.000 ft in Span # 1  
 Fb : Allowable : 1,322.50 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.232 : 1**  
 fv : Actual : 47.96 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 207.00 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{H}$   
 Left Support 0.22 0.37  
 Right Support 0.22 0.37



Max Deflections

Transient Downward	0.584 in	Total Downward	0.934 in
Ratio	452	Ratio	282
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 6. New Roof Joists**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2x8, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Hem-Fir Wood Grade : No.2  
 Fb - Tension 850.0 psi Fc - Prll 1,300.0 psi Fv 150.0 psi Ebend- xx 1,300.0 ksi Density 26.840 pcf  
 Fb - Compr 850.0 psi Fc - Perp 405.0 psi Ft 525.0 psi Eminbend - xx 470.0 ksi

Applied Loads

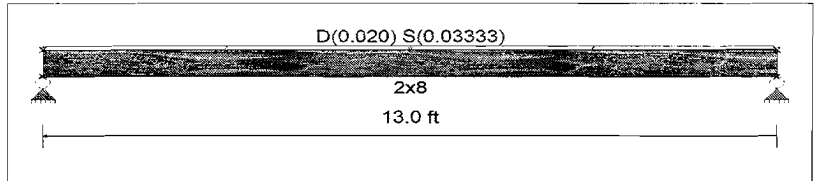
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 1.333 ft

Design Summary

Max fb/Fb Ratio = 0.763 : 1  
 fb : Actual : 1,028.61 psi at 6.500 ft in Span # 1  
 Fb : Allowable : 1,348.95 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.253 : 1  
 fv : Actual : 43.66 psi at 12.437 ft in Span # 1  
 Fv : Allowable : 172.50 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 0.13 0.22  
 Right Support 0.13 0.22



Max Deflections

Transient Downward	0.348 in	Total Downward	0.556 in
Ratio	448	Ratio	280
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design : 7. New Roof Beam at outlooker Joists**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2-2x12, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch Wood Grade : No.1  
 Fb - Tension 1,000.0 psi Fc - Prll 1,500.0 psi Fv 180.0 psi Ebend- xx 1,700.0 ksi Density 31.210 pcf  
 Fb - Compr 1,000.0 psi Fc - Perp 625.0 psi Ft 675.0 psi Eminbend - xx 620.0 ksi

Applied Loads

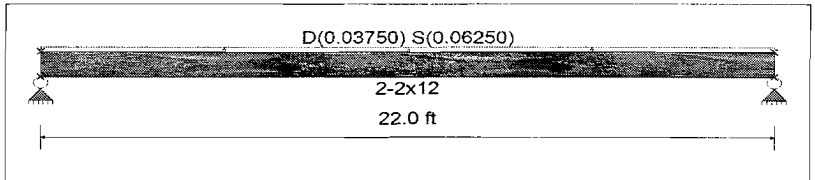
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 2.50 ft

Design Summary

Max fb/Fb Ratio = 0.998 : 1  
 fb : Actual : 1,147.26 psi at 11.000 ft in Span # 1  
 Fb : Allowable : 1,150.00 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.217 : 1  
 fv : Actual : 44.98 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 207.00 psi  
 Load Comb : +D+S

Max Reactions (k) D Lr L S W E H  
 Left Support 0.41 0.69  
 Right Support 0.41 0.69



Max Deflections

Transient Downward	0.547 in	Total Downward	0.876 in
Ratio	482	Ratio	301
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KWV-06015659, Build:20.24.08.01

MYERS ENGINEERING

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**Description :**

**Wood Beam Design :** 7. Roof beam at outlooker joists

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 2-2x8, Sawn, Fully Braced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Hem-Fir

Wood Grade : No.2

Fb - Tension 850.0 psi Fc - Prll 1,300.0 psi Fv 150.0 psi Ebend- xx 1,300.0 ksi Density 26.840 pcf  
 Fb - Compr 850.0 psi Fc - Perp 405.0 psi Ft 525.0 psi Eminbend - xx 470.0 ksi

Applied Loads

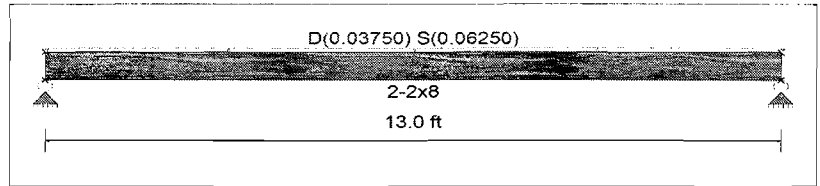
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 2.50 ft

Design Summary

Max fb/Fb Ratio = 0.822 : 1  
 fb : Actual : 964.57 psi at 6.500 ft in Span # 1  
 Fb : Allowable : 1,173.00 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.237 : 1  
 fv : Actual : 40.94 psi at 12.437 ft in Span # 1  
 Fv : Allowable : 172.50 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{H}$   
 Left Support 0.24 0.41  
 Right Support 0.24 0.41



Max Deflections

Transient Downward	0.326 in	Total Downward	0.522 in
Ratio	478	Ratio	299
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design :** 8. Kitchen Floor Joists

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 2x10, Sawn, Fully Braced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch

Wood Grade : No.2

Fb - Tension 900 psi Fc - Prll 1350 psi Fv 180 psi Ebend- xx 1600 ksi Density 31.21 pcf  
 Fb - Compr 900 psi Fc - Perp 625 psi Ft 575 psi Eminbend - xx 580 ksi

Applied Loads

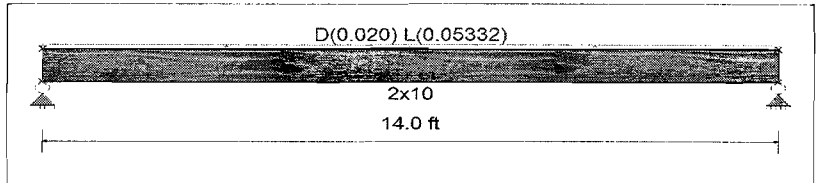
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.333 ft

Design Summary

Max fb/Fb Ratio = 0.885 : 1  
 fb : Actual : 1,007.67 psi at 7.000 ft in Span # 1  
 Fb : Allowable : 1,138.50 psi  
 Load Comb : +D+L

Max fv/FvRatio = 0.275 : 1  
 fv : Actual : 49.56 psi at 13.253 ft in Span # 1  
 Fv : Allowable : 180.00 psi  
 Load Comb : +D+L

Max Reactions (k)  $\frac{D}{L_r}$   $\frac{L}{S}$   $\frac{W}{E}$   $\frac{H}{H}$   
 Left Support 0.14 0.37  
 Right Support 0.14 0.37



Max Deflections

Transient Downward	0.293 in	Total Downward	0.402 in
Ratio	573	Ratio	417
LC: L Only		LC: +D+L	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 9. Rim beam at South wall**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **5.5x10.5, GLB, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

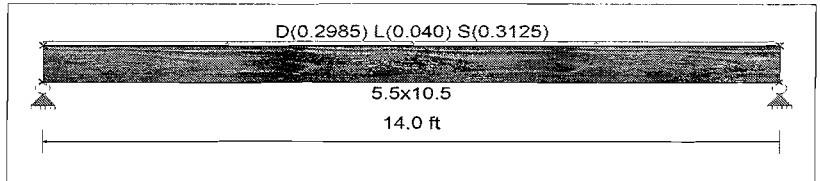
Unif Load: D = 0.2985, L = 0.040, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.644 : 1**  
 fb : Actual : 1,777.45 psi at 7.000 ft in Span # 1  
 Fb : Allowable : 2,760.00 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.321 : 1**  
 fv : Actual : 97.76 psi at 13.160 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+S

Max Reactions (k)	<u>D</u>	<u>Lr</u>	<u>L</u>	<u>S</u>	<u>W</u>	<u>E</u>	<u>H</u>
Left Support	2.09		0.28	2.19			
Right Support	2.09		0.28	2.19			



Max Deflections

Transient Downward	0.284 in	Total Downward	0.556 in
Ratio	590	Ratio	302
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam**

Project File: Long Addition.ec6

LIC#: KVV-06015659, Build:20.24.08.01

MYERS ENGINEERING

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**DESCRIPTION:** 10. Cantilever Rim beam

**CODE REFERENCES**

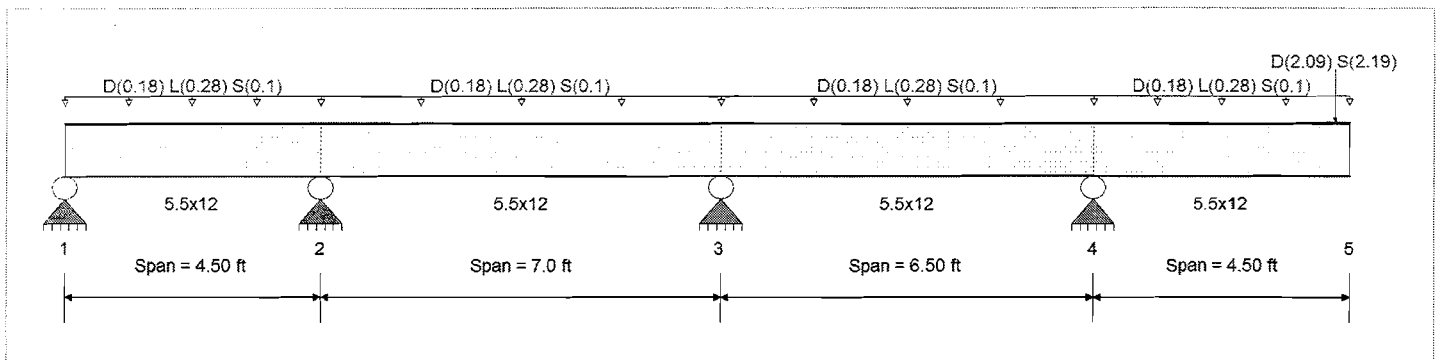
Calculations per NDS 2018, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

**Material Properties**

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	<i>E : Modulus of Elasticity</i>
Load Combination : IBC 2021	Fb -	1,850.0 psi	Ebend- xx
	Fc - Prll	1,650.0 psi	Eminbend - xx
Wood Species : DF/DF	Fc - Perp	650.0 psi	Ebend- yy
Wood Grade : 24F-V4	Fv	265.0 psi	Eminbend - yy
	Ft	1,100.0 psi	Density
			31.210pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



**Applied Loads**

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.180, L = 0.280, S = 0.10, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.180, L = 0.280, S = 0.10, Tributary Width = 1.0 ft

Load for Span Number 3

Uniform Load : D = 0.180, L = 0.280, S = 0.10, Tributary Width = 1.0 ft

Load for Span Number 4

Uniform Load : D = 0.180, L = 0.280, S = 0.10, Tributary Width = 1.0 ft

Point Load : D = 2.090, S = 2.190 k @ 4.250 ft, (Beam 9)

**DESIGN SUMMARY**

**Design OK**

Maximum Bending Stress Ratio	=	0.898	1	Maximum Shear Stress Ratio	=	0.401	: 1
Section used for this span		5.5x12		Section used for this span		5.5x12	
fb: Actual	=	1,911.36psi		fv: Actual	=	122.15 psi	
F'b	=	2,127.50psi		F'v	=	304.75 psi	
Load Combination		+D+S		Load Combination		+D+0.750L+0.750S	
Location of maximum on span	=	6.500ft		Location of maximum on span	=	6.500 ft	
Span # where maximum occurs	=	Span # 3		Span # where maximum occurs	=	Span # 3	
Maximum Deflection							
Max Downward Transient Deflection		0.182 in	Ratio = 592 >= 360	Span: 4 : S Only			
Max Upward Transient Deflection		-0.025 in	Ratio = 3104 >= 360	Span: 3 : S Only			
Max Downward Total Deflection		0.367 in	Ratio = 294 >= 240	Span: 4 : +D+S			
Max Upward Total Deflection		-0.050 in	Ratio = 1553 >= 240	Span: 3 : +D+S			

**Vertical Reactions**

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Max Upward from all Load Conditions	0.465	4.264	1.478	10.979	
Max Upward from Load Combinations	0.465	4.264	0.272	10.979	
Max Upward from Load Cases	0.379	1.933	1.478	5.427	
Max Downward from all Load Conditions	-0.029		-2.938		
Max Downward from Load Combinations			-2.938		

Project Title: Long Addition & Remodel  
 Engineer: Mark Myers, PE  
 Project ID:  
 Project Descr: SFR Addition

**Wood Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

MYERS ENGINEERING

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**DESCRIPTION:** 10. Cantilever Rim beam

**Vertical Reactions**

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Max Downward from Load Cases (Resis	-0.029		-1.732		
D Only	0.086	1.833	-1.206	5.427	
+D+L	0.465	3.766	0.272	7.937	
+D+S	0.057	3.141	-2.938	10.320	
+D+0.750L	0.370	3.283	-0.098	7.310	
+D+0.750L+0.750S	0.348	4.264	-1.397	10.979	
+0.60D	0.052	1.100	-0.724	3.256	
L Only	0.379	1.933	1.478	2.510	
S Only	-0.029	1.308	-1.732	4.893	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

MYERS ENGINEERING

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**Description :**

**Wood Beam Design :** 11. Deck Joists

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 2x10, Sawn, Fully Braced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

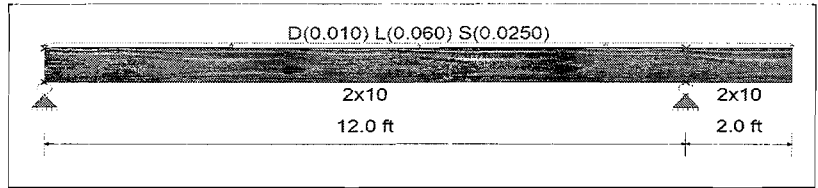
Wood Species: Hem-Fir Wood Grade: No.2  
 Fb - Tension 850.0 psi Fc - Prll 1,300.0 psi Fv 150.0 psi Ebend- xx 1,300.0 ksi Density 26.840 pcf  
 Fb - Compr 850.0 psi Fc - Perp 405.0 psi Ft 525.0 psi Eminbend - xx 470.0 ksi

Applied Loads

Unif Load: D = 0.010, L = 0.060, S = 0.0250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.777 : 1  
 fb : Actual : 668.12 psi at 5.820 ft in Span # 1  
 Fb : Allowable : 860.20 psi  
 Load Comb : +D+L  
 Max fv/FvRatio = 0.343 : 1  
 fv : Actual : 41.22 psi at 11.280 ft in Span # 1  
 Fv : Allowable : 120.00 psi  
 Load Comb : +D+L



Max Reactions (k) D Lr L S W E H  
 Left Support 0.06 0.35 0.15  
 Right Support 0.08 0.49 0.20

Max Deflections

Transient Downward 0.205 in Total Downward 0.252 in  
 Ratio 703 Ratio 571  
 LC: L Only LC: +D+0.750L+0.750S  
 Transient Upward -0.102 in Total Upward -0.125 in  
 Ratio 472 Ratio 384  
 LC: L Only LC: +D+0.750L+0.750S

**Wood Beam Design :** 12. Deck Beam

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size :** 6x12, Sawn, Fully Braced

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

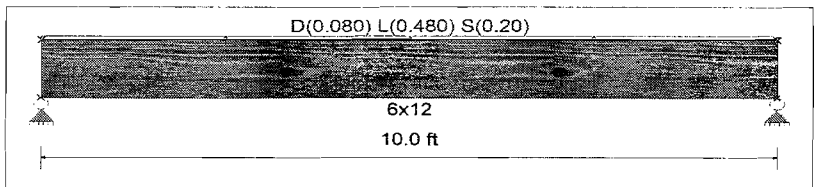
Wood Species: Hem-Fir Wood Grade: No.1  
 Fb - Tension 1050 psi Fc - Prll 750 psi Fv 140 psi Ebend- xx 1300 ksi Density 26.84 pcf  
 Fb - Compr 1050 psi Fc - Perp 405 psi Ft 525 psi Eminbend - xx 470 ksi

Applied Loads

Unif Load: D = 0.080, L = 0.480, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.825 : 1  
 fb : Actual : 692.90 psi at 5.000 ft in Span # 1  
 Fb : Allowable : 840.00 psi  
 Load Comb : +D+L  
 Max fv/FvRatio = 0.482 : 1  
 fv : Actual : 54.01 psi at 9.067 ft in Span # 1  
 Fv : Allowable : 112.00 psi  
 Load Comb : +D+L



Max Reactions (k) D Lr L S W E H  
 Left Support 0.40 2.40 1.00  
 Right Support 0.40 2.40 1.00

Max Deflections

Transient Downward 0.120 in Total Downward 0.147 in  
 Ratio 1001 Ratio 814  
 LC: L Only LC: +D+0.750L+0.750S  
 Transient Upward 0.000 in Total Upward 0.000 in  
 Ratio 9999 Ratio 9999  
 LC: LC:

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 13. Header at 8ft SGD**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **3.5x9, GLB, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension 2400 psi Fc - Prll 1650 psi Fv 265 psi Ebend- xx 1800 ksi Density 31.21 pcf  
 Fb - Compr 1850 psi Fc - Perp 650 psi Ft 1100 psi Eminbend - xx 950 ksi

Applied Loads

Unif Load: D = 0.4815, L = 0.10, S = 0.3375 k/ft, 5.0 ft to 8.0 ft, Trib= 1.0 ft

1Point: D = 0.660, S = 1.090 k @ 5.0 ft

2Point: D = 0.40, L = 2.40, S = 1.0 k @ 6.50 ft

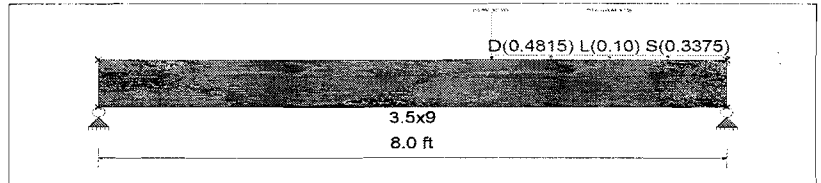
Design Summary

Max fb/Fb Ratio = **0.731 : 1**  
 fb : Actual : 1,985.38 psi at 5.093 ft in Span # 1  
 Fb : Allowable : 2,715.35 psi  
 Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.733 : 1**  
 fv : Actual : 223.30 psi at 7.253 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+0.750L+0.750S

Max Reactions (k)  $\frac{D}{0.59}$   $\frac{Lr}{1.91}$   $\frac{L}{2.19}$   $\frac{S}{2.32}$   $\frac{W}{0.79}$   $\frac{E}{2.32}$   $\frac{H}{0.79}$

Left Support  
 Right Support



Max Deflections

Transient Downward	0.100 in	Total Downward	0.204 in
Ratio	957	Ratio	469
LC: S Only		LC: +D+0.750L+0.750S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design : 14. Floor beam over expanded Bathroom**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **2-1.75x9.25, Microllam LVL, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist

Wood Grade : MicroLam LVL 1.9 E

Fb - Tension 2,600.0 psi Fc - Prll 2,510.0 psi Fv 285.0 psi Ebend- xx 1,900.0 ksi Density 42.010 pcf  
 Fb - Compr 2,600.0 psi Fc - Perp 750.0 psi Ft 1,555.0 psi Eminbend - xx 965.71 ksi

Applied Loads

Unif Load: D = 0.3375, S = 0.5625 k/ft, Trib= 1.0 ft

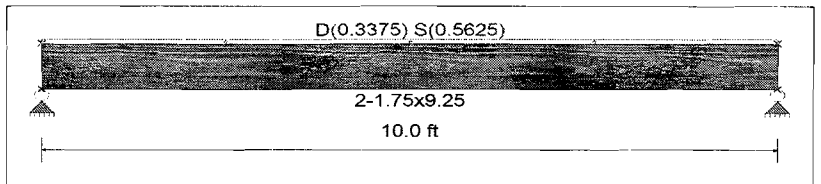
Design Summary

Max fb/Fb Ratio = **0.907 : 1**  
 fb : Actual : 2,704.79 psi at 5.000 ft in Span # 1  
 Fb : Allowable : 2,983.28 psi  
 Load Comb : +D+S

Max fv/FvRatio = **0.539 : 1**  
 fv : Actual : 176.53 psi at 9.233 ft in Span # 1  
 Fv : Allowable : 327.75 psi  
 Load Comb : +D+S

Max Reactions (k)  $\frac{D}{1.69}$   $\frac{Lr}{1.69}$   $\frac{L}{2.81}$   $\frac{S}{2.81}$   $\frac{W}{2.81}$   $\frac{E}{2.81}$   $\frac{H}{2.81}$

Left Support  
 Right Support



Max Deflections

Transient Downward	0.290 in	Total Downward	0.464 in
Ratio	413	Ratio	258
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

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**Wood Beam Design : 15. Floor beam below Mudroom wall supporting Beam 1**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size : 4-1.75x9.25, Microllam LVL, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

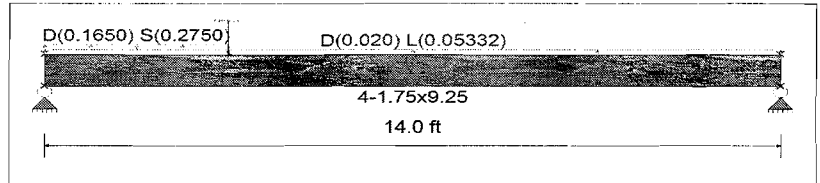
Wood Species: iLevel Truss Joist Wood Grade: MicroLam LVL 1.9 E  
 Fb - Tension 2,600.0 psi Fc - Prll 2,510.0 psi Fv 285.0 psi Ebend- xx 1,900.0 ksi Density 42.010 pcf  
 Fb - Compr 2,600.0 psi Fc - Perp 750.0 psi Ft 1,555.0 psi Eminbend - xx 965.71 ksi

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.333 ft  
 Unif Load: D = 0.1650, S = 0.2750 k/ft, 0.0 to 3.50 ft, Trib= 1.0 ft  
 1Point: D = 2.10, S = 3.510 k @ 3.50 ft

Design Summary

Max fb/Fb Ratio = 0.670 : 1  
 fb : Actual : 2,057.43 psi at 3.500 ft in Span # 1  
 Fb : Allowable : 3,069.78 psi  
 Load Comb : +D+S  
 Max fv/FvRatio = 0.378 : 1  
 fv : Actual : 123.97 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 327.75 psi  
 Load Comb : +D+S



Max Reactions (k)  $\frac{D}{L}$   $\frac{L}{S}$   $\frac{S}{W}$   $\frac{W}{E}$   $\frac{E}{H}$   
 Left Support 2.22 0.37 3.47  
 Right Support 0.74 0.37 1.00

Max Deflections

Transient Downward	0.318 in	Total Downward	0.528 in
Ratio	528	Ratio	318
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Wood Beam Design : 15a. Floor beam below Mudroom wall**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

**BEAM Size : 1.75x9.25, Microllam LVL, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

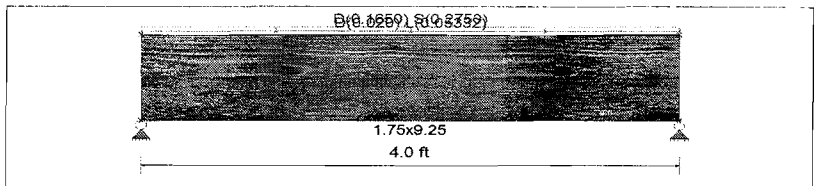
Wood Species: iLevel Truss Joist Wood Grade: MicroLam LVL 1.9 E  
 Fb - Tension 2,600.0 psi Fc - Prll 2,510.0 psi Fv 285.0 psi Ebend- xx 1,900.0 ksi Density 42.010 pcf  
 Fb - Compr 2,600.0 psi Fc - Perp 750.0 psi Ft 1,555.0 psi Eminbend - xx 965.71 ksi

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.333 ft  
 Unif Load: D = 0.1650, S = 0.2750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.160 : 1  
 fb : Actual : 442.38 psi at 2.000 ft in Span # 1  
 Fb : Allowable : 2,758.53 psi  
 Load Comb : +D+S  
 Max fv/FvRatio = 0.161 : 1  
 fv : Actual : 52.86 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 327.75 psi  
 Load Comb : +D+S



Max Reactions (k)  $\frac{D}{L}$   $\frac{L}{S}$   $\frac{S}{W}$   $\frac{W}{E}$   $\frac{E}{H}$   
 Left Support 0.37 0.11 0.55  
 Right Support 0.37 0.11 0.55

Max Deflections

Transient Downward	0.007 in	Total Downward	0.012 in
Ratio	6610	Ratio	3951
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Multiple Simple Beam**

Project File: Long Addition.ec6

LIC#: KW-06015659, Build:20.24.08.01

MYERS ENGINEERING

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**Wood Beam Design : 16. Header at Storage**

Calculations per NDS 2018, IBC 2021, ASCE 7-16

BEAM Size : **3.5x9, GLB, Fully Unbraced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF

Wood Grade : 24F-V4

Fb - Tension 2400 psi Fc - Prll 1650 psi Fv 265 psi Ebend-xx 1800 ksi Density 31.21 pcf  
 Fb - Compr 1850 psi Fc - Perp 650 psi Ft 1100 psi Eminbend - xx 950 ksi

Applied Loads

Unif Load: D = 0.1350, L = 0.360 k/ft, Trib= 1.0 ft

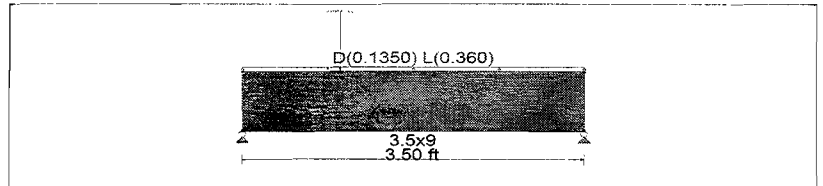
1Point: D = 2.590, S = 4.020 k @ 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.452 : 1  
 fb : Actual : 1,240.44 psi at 1.003 ft in Span # 1  
 Fb : Allowable : 2,741.76 psi  
 Load Comb : +D+S

Max fv/FvRatio = 0.759 : 1  
 fv : Actual : 231.28 psi at 0.000 ft in Span # 1  
 Fv : Allowable : 304.75 psi  
 Load Comb : +D+S

Max Reactions (k)	<u>D</u>	<u>Lr</u>	<u>L</u>	<u>S</u>	<u>W</u>	<u>E</u>	<u>H</u>
Left Support	2.09		0.63	2.87			
Right Support	0.98		0.63	1.15			



Max Deflections

Transient Downward	0.013 in	Total Downward	0.022 in
Ratio	3325	Ratio	1913
LC: S Only		LC: +D+S	
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
LC:		LC:	

**Maximum Load For 6x6 DF#1 Wood Post**

$$\frac{\text{psf}}{\text{ft}} := \frac{\text{psi}}{144} \quad \frac{\text{plf}}{\text{ft}} := \text{psf} \cdot \text{ft} \quad \frac{\text{lb}}{\text{ft}} := \text{plf} \cdot \text{ft} \quad H := 8 \cdot \text{ft}$$

$$F_c := 1000 \cdot \text{psi} \quad C_D := 1 \quad C_{Fb} := 1 \quad C_M := 1 \quad C_w := 1 \quad C_L := 1 \quad C_{Fc} := 1$$

$$E' := 1600000 \cdot \text{psi}$$

$$F'_c := F_c \cdot C_D \cdot C_{Fc} \quad F'_c = 1000 \cdot \text{psi}$$

Axial Load Capacity

Slenderness Ratio (SL)

$$SL := \frac{H}{h} \quad C := 0.8 \quad K_{CE} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} \quad F_{CE} = 1576 \cdot \text{psi}$$

$$C_p := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c}} \right] \cdot K_f$$

$$F'_c := C_p \cdot F'_c \quad F'_c = 821 \cdot \text{psi} \quad P_{\text{max}} := F'_c \cdot A \quad P_{\text{max}} = 24841 \cdot \text{lb} \quad (\text{Maximum post Capacity})$$

**6x6 Wood Post Properties**

$$K_f := 1 \quad (K_f = 0.6 \text{ for unbraced nailed built up posts} - 0.75 \text{ for bolted})$$

$$h := 5.5 \cdot \text{in}$$

$$t := 5.5 \cdot \text{in}$$

$$A := t \cdot h \quad A = 30.2 \cdot \text{in}^2$$

$$I := \frac{t \cdot h^3}{12} \quad I = 76.3 \cdot \text{in}^4$$

$$S := \frac{I \cdot 2}{h} \quad S = 27.7 \cdot \text{in}^3$$

$$C_p = 0.82$$

**Maximum Load For 6x6 HF#2 Treated Post**

$$\frac{\text{psf}}{\text{ft}} := \frac{\text{psi}}{144} \quad \frac{\text{plf}}{\text{ft}} := \text{psf} \cdot \text{ft} \quad \frac{\text{lb}}{\text{ft}} := \text{plf} \cdot \text{ft} \quad H := 8 \cdot \text{ft}$$

$$F_c := 460 \cdot \text{psi} \quad C_D := 1 \quad C_{Fb} := 1 \quad C_M := 1 \quad C_w := 1 \quad C_L := 1 \quad C_{Fc} := 1$$

$$E' := 1045000 \cdot \text{psi}$$

$$F'_c := F_c \cdot C_D \cdot C_{Fc} \quad F'_c = 460 \cdot \text{psi}$$

Axial Load Capacity

Slenderness Ratio (SL)

$$SL := \frac{H}{h} \quad C := 0.8 \quad K_{CE} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} \quad F_{CE} = 1029 \cdot \text{psi}$$

$$C_p := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c}} \right] \cdot K_f$$

$$F'_c := C_p \cdot F'_c \quad F'_c = 407 \cdot \text{psi} \quad P_{\text{max}} := F'_c \cdot A \quad P_{\text{max}} = 12306 \cdot \text{lb} \quad (\text{Maximum post Capacity})$$

**6x6 Treated Wood Post Properties**

$$K_f := 1.0 \quad (K_f = 0.6 \text{ for unbraced nailed built up posts} - 0.75 \text{ for bolted})$$

$$h := 5.5 \cdot \text{in}$$

$$t := 5.5 \cdot \text{in}$$

$$A := t \cdot h \quad A = 30.2 \cdot \text{in}^2$$

$$I := \frac{t \cdot h^3}{12} \quad I = 76.3 \cdot \text{in}^4$$

$$S := \frac{I \cdot 2}{h} \quad S = 27.7 \cdot \text{in}^3$$

$$C_p = 0.88$$

**Maximum Load For 3-2x6 HF Stud Built up Wood Post**

$$\frac{\text{psf}}{\text{ft}} := \frac{\text{psi}}{144} \quad \frac{\text{plf}}{\text{ft}} := \text{psf} \cdot \text{ft} \quad \frac{\text{lb}}{\text{ft}} := \text{plf} \cdot \text{ft} \quad H := 10 \cdot \text{ft}$$

$$F_c := 800 \cdot \text{psi} \quad C_D := 1 \quad C_{FB} := 1 \quad C_M := 1 \quad C_t := 1 \quad C_L := 1 \quad C_{FEM} := 1.1$$

$$E' := 1200000 \cdot \text{psi}$$

$$F'_c := F_c \cdot C_D \cdot C_{Fc} \quad F'_c = 880 \cdot \text{psi}$$

Axial Load Capacity

Slenderness Ratio (SL)

$$SL := \frac{H}{h} \quad C := 0.8 \quad K_{CE} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} \quad F_{CE} = 756 \cdot \text{psi}$$

$$C_{PA} := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c} \cdot \frac{1}{C}} \right] \cdot K_f$$

$$F'_p := C_p \cdot F'_c \quad F'_c = 560 \cdot \text{psi} \quad P_{max} := F'_c \cdot A$$

**3-2x6 Built Up Post Properties**

$$K_f := 1.0 \quad (K_f = 0.6 \text{ for unbraced nailed built up posts} - 0.75 \text{ for bolted})$$

$$h := (5.5) \cdot \text{in}$$

$$t := 3 \cdot (1.5) \cdot \text{in}$$

$$A := t \cdot h \quad A = 24.8 \cdot \text{in}^2$$

$$I := \frac{t \cdot h^3}{12} \quad I = 62.4 \cdot \text{in}^4$$

$$S := \frac{I \cdot 2}{h} \quad S = 22.7 \cdot \text{in}^3$$

$$C_p = 0.64$$

$$P_{max} = 13863 \cdot \text{lb} \quad (\text{Maximum post Capacity})$$

**Maximum Load For 2-2x6 HF Stud Built up Wood Post**

$$\frac{\text{psf}}{\text{ft}} := \frac{\text{psi}}{144} \quad \frac{\text{plf}}{\text{ft}} := \text{psf} \cdot \text{ft} \quad \frac{\text{lb}}{\text{ft}} := \text{plf} \cdot \text{ft} \quad H := 10 \cdot \text{ft}$$

$$F_c := 800 \cdot \text{psi} \quad C_D := 1 \quad C_{FB} := 1 \quad C_M := 1 \quad C_t := 1 \quad C_L := 1 \quad C_{FEM} := 1.1$$

$$E' := 1200000 \cdot \text{psi}$$

$$F'_c := F_c \cdot C_D \cdot C_{Fc} \quad F'_c = 880 \cdot \text{psi}$$

Axial Load Capacity

Slenderness Ratio (SL)

$$SL := \frac{H}{h} \quad C := 0.8 \quad K_{CE} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} \quad F_{CE} = 756 \cdot \text{psi}$$

$$C_{PA} := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c} \cdot \frac{1}{C}} \right] \cdot K_f$$

$$F'_p := C_p \cdot F'_c \quad F'_c = 560 \cdot \text{psi} \quad P_{max} := F'_c \cdot A$$

**2-2x6 Built Up Post Properties**

$$K_f := 1.0 \quad (K_f = 0.6 \text{ for unbraced nailed built up posts} - 0.75 \text{ for bolted})$$

$$h := 5.5 \cdot \text{in}$$

$$t := (2) \cdot 1.5 \cdot \text{in}$$

$$A := t \cdot h \quad A = 16.5 \cdot \text{in}^2$$

$$I := \frac{t \cdot h^3}{12} \quad I = 41.6 \cdot \text{in}^4$$

$$S := \frac{I \cdot 2}{h} \quad S = 15.1 \cdot \text{in}^3$$

$$C_p = 0.64$$

$$P_{max} = 9242 \cdot \text{lb} \quad (\text{Maximum post Capacity})$$

**Maximum Load For 3-2x4 HF Stud Built up Wood Post**

$\frac{\text{psf}}{\text{max}} := \frac{\text{psi}}{144}$      $\frac{\text{plf}}{\text{max}} := \text{psf} \cdot \text{ft}$      $\frac{\text{lb}}{\text{max}} := \text{plf} \cdot \text{ft}$      $\frac{H}{\text{max}} := 10\text{-ft}$

$F_c := 800\text{-psi}$      $C_D := 1$      $C_{FB} := 1$      $C_M := 1$      $C_t := 1$      $C_L := 1$      $C_{FC} := 1.1$

$E' := 1200000\text{-psi}$

$F'_c := F_c \cdot C_D \cdot C_{FC}$      $F'_c = 880\text{-psi}$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$      $C := 0.8$      $K_{CE} := 0.3$

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$      $F_{CE} = 306\text{-psi}$

$$C_p := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c}} \right] \cdot K_f$$

$F'_p := C_p \cdot F'_c$

$F'_c = 280\text{-psi}$

$P_{\text{max}} := F'_c \cdot A$

$P_{\text{max}} = 4411\text{-lb}$  (Maximum post Capacity)

**3-2x4 Built Up Post Properties**

$K_f := 1.0$     ( $K_f = 0.6$  for unbraced nailed built up posts - 0.75 for bolted)

$h := 3.5\text{-in}$

$t := 3 \cdot 1.5\text{-in}$

$A := t \cdot h$      $A = 15.7\text{-in}^2$

$I := \frac{t \cdot h^3}{12}$      $I = 16.1\text{-in}^4$

$S := \frac{I \cdot 2}{h}$      $S = 9.2\text{-in}^3$

$C_p = 0.32$

**Maximum Load For 2-2x4 HF Stud Built up Wood Post**

$\frac{\text{psf}}{\text{max}} := \frac{\text{psi}}{144}$      $\frac{\text{plf}}{\text{max}} := \text{psf} \cdot \text{ft}$      $\frac{\text{lb}}{\text{max}} := \text{plf} \cdot \text{ft}$      $\frac{H}{\text{max}} := 10\text{-ft}$

$F_c := 800\text{-psi}$      $C_D := 1$      $C_{FB} := 1$      $C_M := 1$      $C_t := 1$      $C_L := 1$      $C_{FC} := 1.1$

$E' := 1200000\text{-psi}$

$F'_c := F_c \cdot C_D \cdot C_{FC}$      $F'_c = 880\text{-psi}$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$      $C := 0.8$      $K_{CE} := 0.3$

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$      $F_{CE} = 306\text{-psi}$

$$C_p := \left[ \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F'_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F'_c}} \right] \cdot K_f$$

$F'_p := C_p \cdot F'_c$

$F'_c = 280\text{-psi}$

$P_{\text{max}} := F'_c \cdot A$

$P_{\text{max}} = 2941\text{-lb}$  (Maximum post Capacity)

**2-2x4 Built Up Post Properties**

$K_f := 1.0$     ( $K_f = 0.6$  for unbraced nailed built up posts - 0.75 for bolted)

$h := 3.5\text{-in}$

$t := (2) \cdot 1.5\text{-in}$

$A := t \cdot h$      $A = 10.5\text{-in}^2$

$I := \frac{t \cdot h^3}{12}$      $I = 10.7\text{-in}^4$

$S := \frac{I \cdot 2}{h}$      $S = 6.1\text{-in}^3$

$C_p = 0.32$

**Maximum Load For 4x4 HF#2 Treated Post**

$\frac{\text{psf}}{\text{ft}} := \frac{\text{psi}}{144}$      $\frac{\text{plf}}{\text{ft}} := \text{psf} \cdot \text{ft}$      $\frac{\text{lb}}{\text{ft}} := \text{plf} \cdot \text{ft}$      $H := 8 \cdot \text{ft}$

$F_w := 1040 \cdot \text{psi}$      $C_D := 1$      $C_{Fb} := 1$      $C_M := 1$      $C_{t1} := 1$      $C_{L1} := 1$      $C_{Fc} := 1$

$E' := 1235000 \cdot \text{psi}$

$F_w'' := F_c \cdot C_D \cdot C_{Fc}$      $F_c'' = 1040 \cdot \text{psi}$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$      $C := 0.8$      $K_{CE} := 0.3$

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$      $F_{CE} = 492 \cdot \text{psi}$

$$C_p := \left[ \frac{1 + \frac{F_{CE}}{F_c''}}{2 \cdot C} - \sqrt{\left( \frac{1 + \frac{F_{CE}}{F_c''}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F_c''}} \right] \cdot K_f$$

$F_w' := C_p \cdot F_w''$      $F_c' = 431 \cdot \text{psi}$      $P_{max} := F_c' \cdot A$      $P_{max} = 5284 \cdot \text{lb}$  (Maximum post Capacity)

**4x4 Treated Wood Post Properties**

$K_f := 1.0$     ( $K_f = 0.6$  for unbraced nailed built up posts - 0.75 for bolted)

$h := 3.5 \cdot \text{in}$

$t := 3.5 \cdot \text{in}$

$A := t \cdot h$      $A = 12.2 \cdot \text{in}^2$

$I := \frac{t \cdot h^3}{12}$      $I = 12.5 \cdot \text{in}^4$

$S := \frac{I \cdot 2}{h}$      $S = 7.1 \cdot \text{in}^3$

$C_p = 0.41$