

MYERS ENGINEERING

Lateral & Gravity Calculations



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Project: RKK Spec Residence
4115 78th Avenue SE
Mercer Island, WA

April 3, 2025

2021 INTERNATIONAL BUILDING CODE
100 MPH BASIC WIND, EXPOSURE C, $K_{zt} = 1.61$
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

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

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

WOODS :

WOOD TYPE:

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

GLUED-LAMINATED (GLB) BEAM & HEADER.
 $F_b=2,400$ PSI, $F_v=165$ PSI, F_c (Perp) =650 PSI, $E=1,800,000$ PSI.

PARALLAM (PSL) 2.0E BEAM & HEADER.
 $F_b=2,900$ PSI, $F_v=290$ PSI, F_c (Perp) =750 PSI, $E=2,000,000$ PSI.

MICROLAM (LVL) 1.9E BEAM & HEADER
 $F_b=2,600$ PSI, $F_v=285$ PSI, F_c (Perp) =750 PSI, $E=1,900,000$ PSI.

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD
 $F_b=1,700$ PSI, $F_v=400$ PSI, F_c (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.

ASCE Hazards Report

Address:
4115 78th Ave SE
Mercer Island, Washington
98040

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 47.57253
Longitude: -122.234664
Elevation: 171.98973677991813 ft
(NAVD 88)



Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	74 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Tue Mar 18 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Site Soil Class: D - Stiff Soil

Results:

S_s :	1.422	S_{D1} :	N/A
S_1 :	0.494	T_L :	6
F_a :	1	PGA :	0.609
F_v :	N/A	PGA _M :	0.669
S_{MS} :	1.422	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.948	C_v :	1.384

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Tue Mar 18 2025

Date Source: [USGS Seismic Design Maps](#)

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)

$R := 6.5$ $\Omega_0 := 3.0$ $C_d := 4$

Light-frame (wood) walls sheathed w/ wood structural panels rated for shear resistance (ASCE 7-16 Table 12.2-1)

$S_s := 1.42$

$S_1 := 0.49$

$S_{ms} := 1.42$

$S_{m1} := 0.96$

Equation 11.4-3

$S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.95$

Equation 11.4-4

$S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.64$

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

Roof Adjustment:

Plan Area for Each Level:

$A_1 := 1.2 (1263) \cdot ft^2$

$A_{1a} := 1.2 (905) \cdot ft^2$

$A_2 := 1537 \cdot ft^2$

$A_{2a} := 1146 \cdot ft^2 + 1.2 (140) \cdot ft^2$

(NW Upper Roof)

(SE Upper Roof)

(NW Upper Fir+Low Roof)

(SE Upper Fir + Low Roof)

Slope Factor

4 1.05

5 1.08

6 1.12

7 1.16

8 1.20

9 1.25

10 1.3

11 1.36

12 1.41

Plan Perimeter for Each Level:

$P_1 := 2 \cdot (33 \cdot ft) + 2 \cdot (33 \cdot ft)$

$P_2 := 2 \cdot (28 \cdot ft) + 2 \cdot (33 \cdot ft)$

(NW Upper Floor)

(NW Main Floor)

$P_3 := 2 \cdot (33 \cdot ft) + 2 \cdot (41 \cdot ft)$

$P_4 := 2 \cdot (33 \cdot ft) + 2 \cdot (48 \cdot ft)$

(SE Upper Floor)

(SE Main Floor)

$W_{w,x}$ = Seismic Weight of Overall Structure, Seismic Weight of Structure above Level x (LB.)

Weight of Structure at Each Level:

Story Weight at Upper Floor:

$w_1 := 15 \cdot psf \cdot (A_1) + 12 \cdot psf \cdot 4.5 \cdot ft \cdot (P_1)$

Story Weight at Attic:

$w_3 := 15 \cdot psf \cdot (A_{1a}) + 12 \cdot psf \cdot 4.5 \cdot ft \cdot (P_3)$

Story Weight at Main Floor:

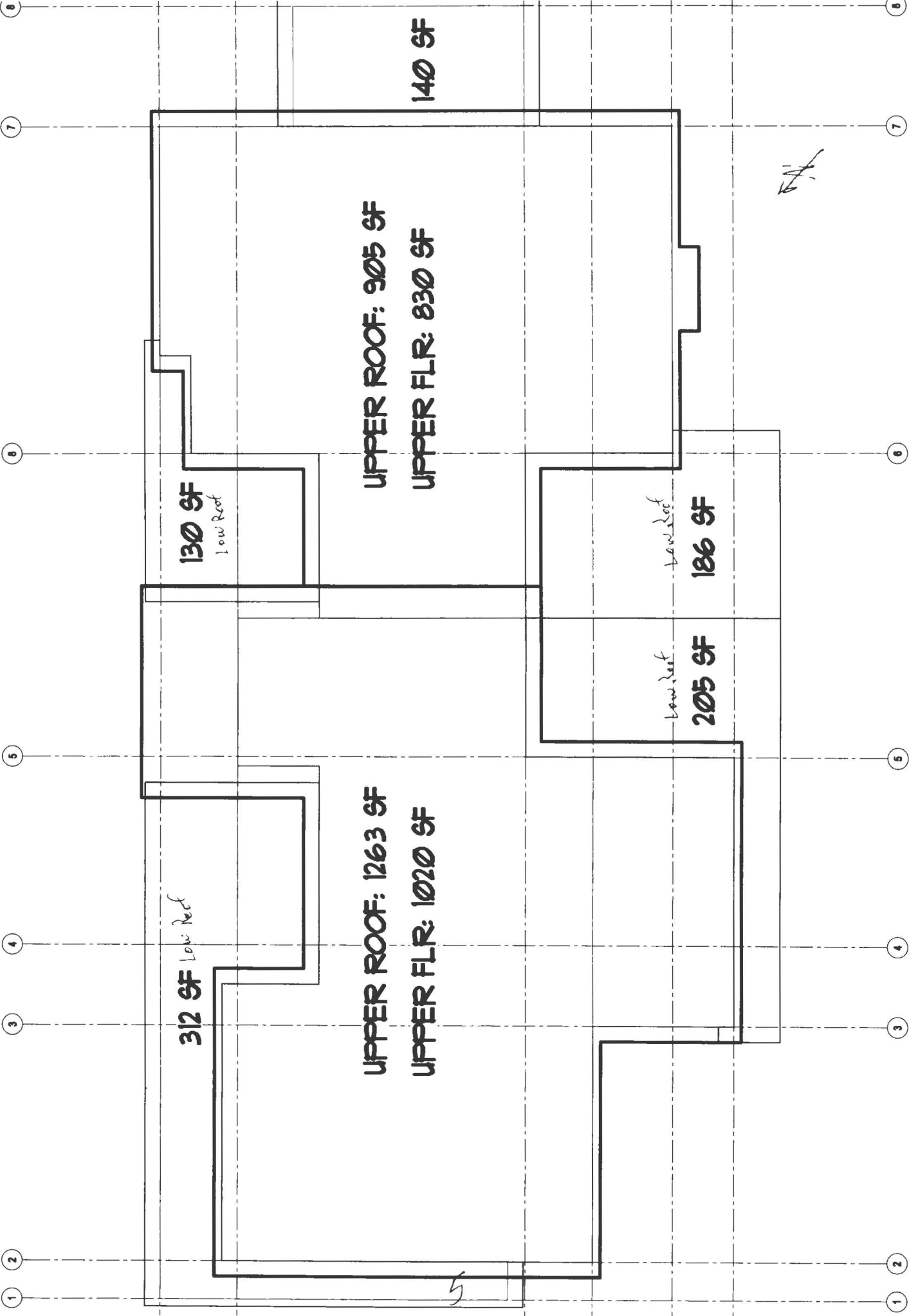
$w_2 := 15 \cdot psf \cdot (A_2) + 12 \cdot psf \cdot (4.5 \cdot ft \cdot P_1 + 5 \cdot ft \cdot P_2)$

Story Weight at Garage:

$w_4 := 15 \cdot psf \cdot (A_{2a}) + 12 \cdot psf \cdot (4.5 \cdot ft \cdot P_3 + 5 \cdot ft \cdot P_4)$

$W_a := w_1 + w_2 = 67365.00 lb$

$W_b := w_3 + w_4 = 61704.00 lb$



312 SF Low Roof

130 SF Low Roof

205 SF Low Roof

186 SF Low Roof

UPPER ROOF: 1263 SF
UPPER FLR: 1020 SF

UPPER ROOF: 905 SF
UPPER FLR: 830 SF

140 SF



Approximate Fundamental Period, T_a :

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

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

T_a is less than T_L , therefore C_s need not exceed: $\frac{S_{DI}}{\left(\frac{R}{I_e}\right) \cdot T_a} = 0.44$ (ASCE 7-16 Eq. 12.8-3)

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

$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.15$ (ASCE 7-16 Eq. 12.8-2)

House Base Shear: $V_a := C_s \cdot (W_a) = 9811.11 \text{ lb}$

Garage Base Shear: $V_b := C_s \cdot (W_b) = 8986.63 \text{ lb}$

Total Base Shear: $V_E := C_s \cdot (W_a + W_b) = 18797.74 \text{ lb}$

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: $k := 1$

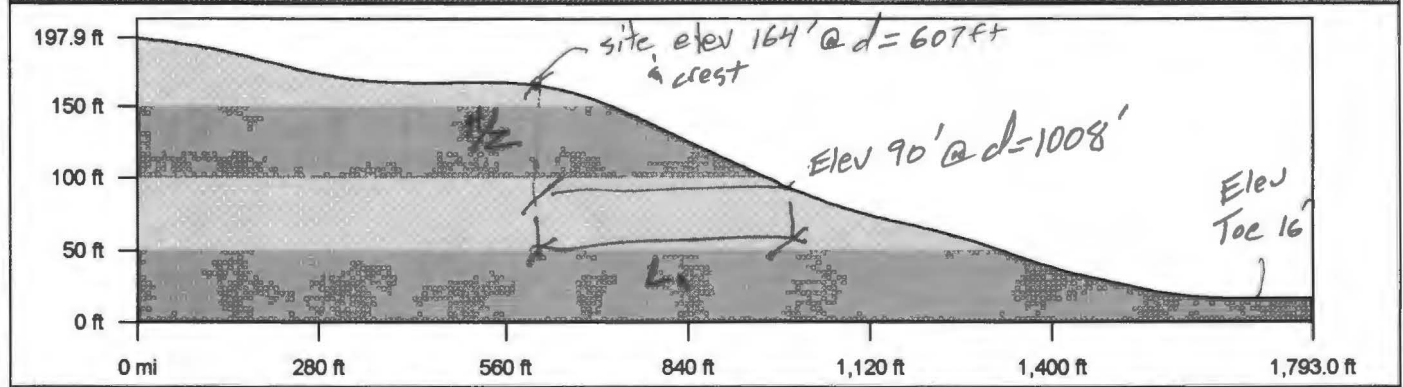
$h_1 := 20 \cdot ft$ $h_2 := 10 \cdot ft$ (Height from base to level x)

$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.61$ $F_1 := C_{v1} \cdot V_a = 6026.71 \text{ lb}$ Story Shear at NW Upper Floor

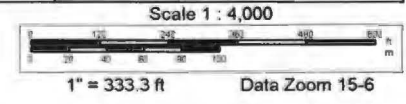
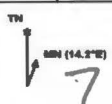
$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.39$ $F_2 := C_{v2} \cdot V_a = 3784.40 \text{ lb}$ Story Shear at NW Main Floor

$C_{v3} := \frac{(w_3 \cdot h_1)}{(w_3 \cdot h_1 + w_4 \cdot h_2)} = 0.56$ $F_3 := C_{v3} \cdot V_b = 5075.56 \text{ lb}$ Story Shear at SE Upper Floor

$C_{v4} := \frac{(w_4 \cdot h_2)}{(w_3 \cdot h_1 + w_4 \cdot h_2)} = 0.44$ $F_4 := C_{v4} \cdot V_b = 3911.08 \text{ lb}$ Story Shear at SE Main Floor



Lin Dist: 1,779.6 ft	Terr Dist: 1,793.0 ft	Elev Gain: -181.6 ft	Avg Grade: 10
Climb Elev: 0.6 ft	Desc Elev: 182.2 ft	Max. Elev: 197.9 ft	Min. Elev: 16.1 ft
Climb Dist: 169.5 ft	Desc Dist: 1,623.5 ft		



WIND DESIGN

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

$V := 100$ Nominal 3-Sec Gust (MPH) for Risk Category II (Figure 26.5-1B).

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 25 \cdot ft$ Mean Roof Height as per Sect. 26.2

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

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

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

$x := 0 \cdot ft$ $H := 148 \cdot ft$ $L_h := 401 \cdot ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.85 \cdot \left(\frac{H}{L_h}\right) = 0.31$$

$$K_2 := \left(1 - \frac{x}{\mu \cdot L_h}\right) = 1.00$$

$$K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.86$$

$$K_{zt} := (1 + K_1 \cdot K_2 \cdot K_3)^2 = 1.61$$

$G := 0.85$ 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):

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

$z_1 := 20 \cdot ft$ $z_2 := 15 \cdot ft$ Height from ground to level x ($z_{min} = 15ft$)

$$K_{z1} := 2.01 \cdot \left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.90$$

$$K_{z2} := 2.01 \cdot \left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.85$$

$$K_h := 2.01 \cdot \left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.95$$

External Pressure Coefficients w/ Roof Pitch = 8/12 (34 degrees) Front to Back & 8/12 (34 degrees) Side to Side
 Taken from Figure 27.3-1

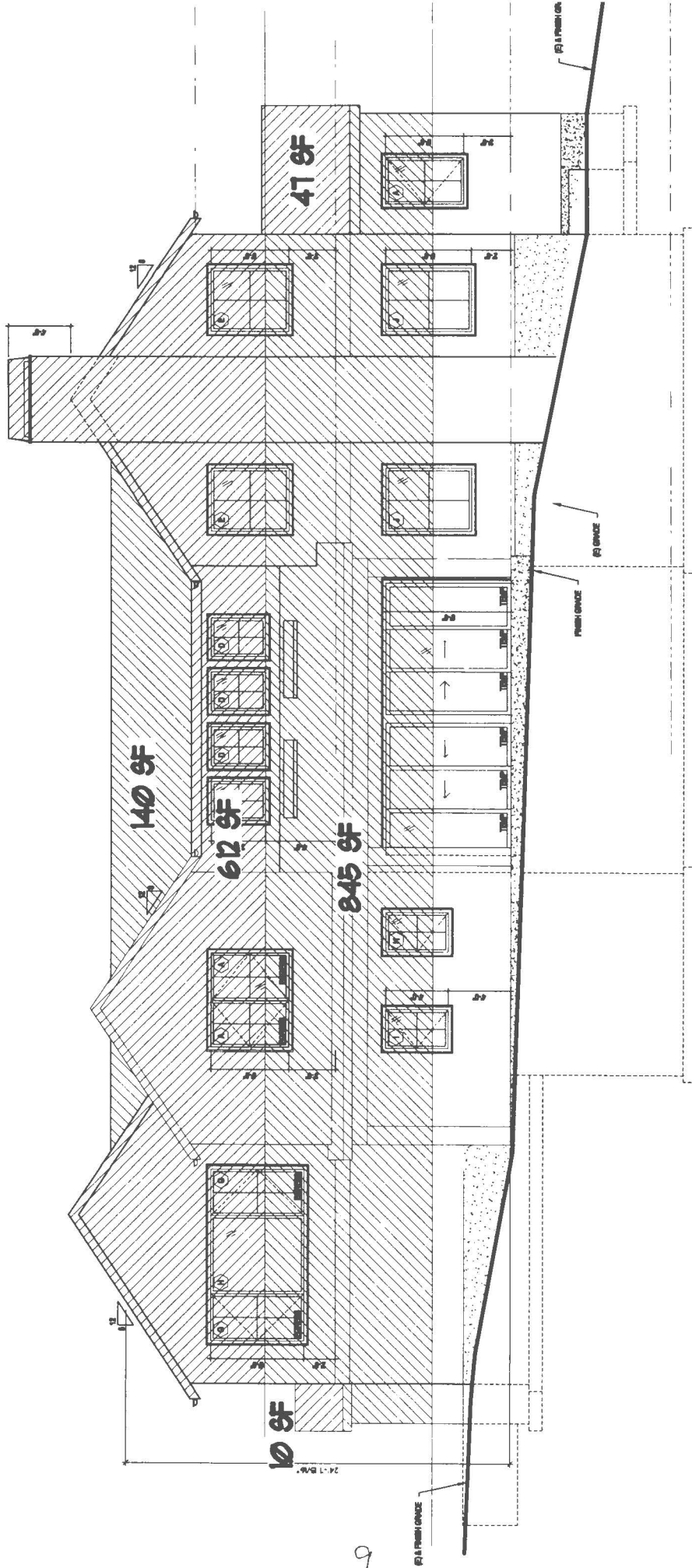
Front to Back:

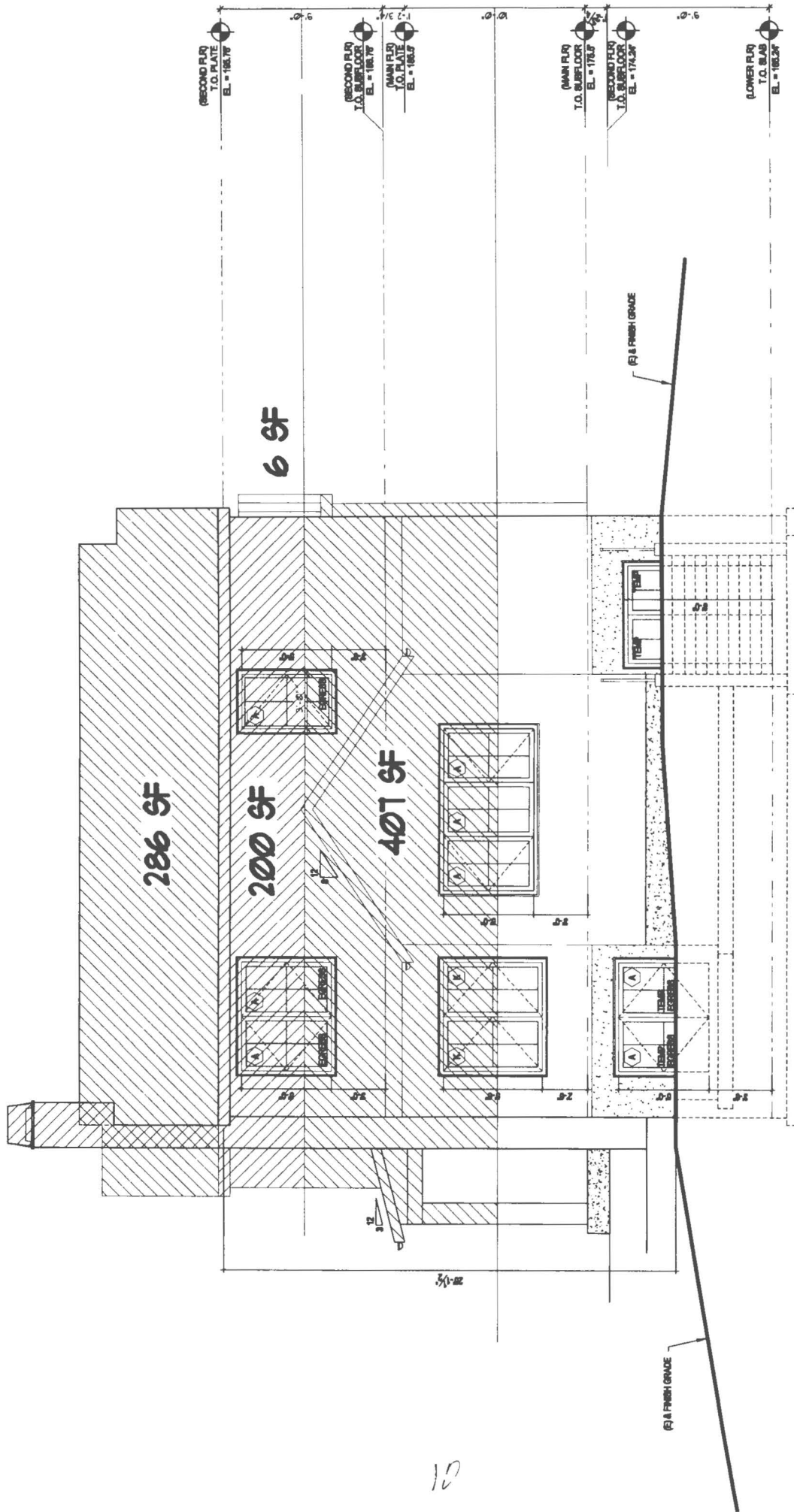
Side to Side:

$L_{fb} := 33 \cdot ft$ $B_{fb} := 74 \cdot ft$ $\frac{L_{fb}}{B_{fb}} = 0.45$ $\frac{h}{L_{fb}} = 0.76$ $L_{ss} := 74 \cdot ft$ $B_{ss} := 33 \cdot ft$ $\frac{L_{ss}}{B_{ss}} = 2.24$ $\frac{h}{L_{ss}} = 0.34$

$C_{pf1} := 0.8$	Windward Wall	$C_{ps1} := 0.8$	Windward Wall
$C_{pr1} := 0.25$	Windward Roof	$C_{ps2} := 0.36$	Windward Roof
$C_{pl1} := -0.6$	Leeward Roof	$C_{ps3} := -0.6$	Leeward Roof
$C_{pw1} := -0.5$	Leeward Wall	$C_{ps4} := -0.29$	Leeward Wall

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

$$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 31.58$$

$$q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 29.72$$

$$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 33.09$$

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 $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 21.47 \frac{lb}{ft^2}$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 20.21 \frac{lb}{ft^2}$

Front to Back Pressures:

Side to Side Pressures:

Windward Roof $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 7.03 \frac{lb}{ft^2}$

Windward Roof $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 10.13 \frac{lb}{ft^2}$

Leeward Roof $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -16.88 \frac{lb}{ft^2}$

Leeward Roof $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -16.88 \frac{lb}{ft^2}$

Leeward Wall $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -14.07 \frac{lb}{ft^2}$

Leeward Wall $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -8.16 \frac{lb}{ft^2}$

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 8psf at roof & 16psf at walls over projected vertical plane per ASCE 7-16 Sec. 27.1-5:

$$p_{wr1} - p_{lr1} = 23.91 \frac{lb}{ft^2}$$

$$p_{ww1} - p_{lw1} = 35.54 \frac{lb}{ft^2}$$

$$p_{ww2} - p_{lw2} = 34.27 \frac{lb}{ft^2}$$

$$p_{wr2} - p_{lr2} = 27.00 \frac{lb}{ft^2}$$

$$p_{ww1} - p_{lw2} = 29.63 \frac{lb}{ft^2}$$

$$p_{ww2} - p_{lw2} = 28.37 \frac{lb}{ft^2}$$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1}) \cdot 170 \cdot ft^2 + (p_{ww1} - p_{lw1}) \cdot 265 \cdot ft^2 = 13481.93 \text{ lb}$$

Wind Pressure at Main Floor (Front to Back):

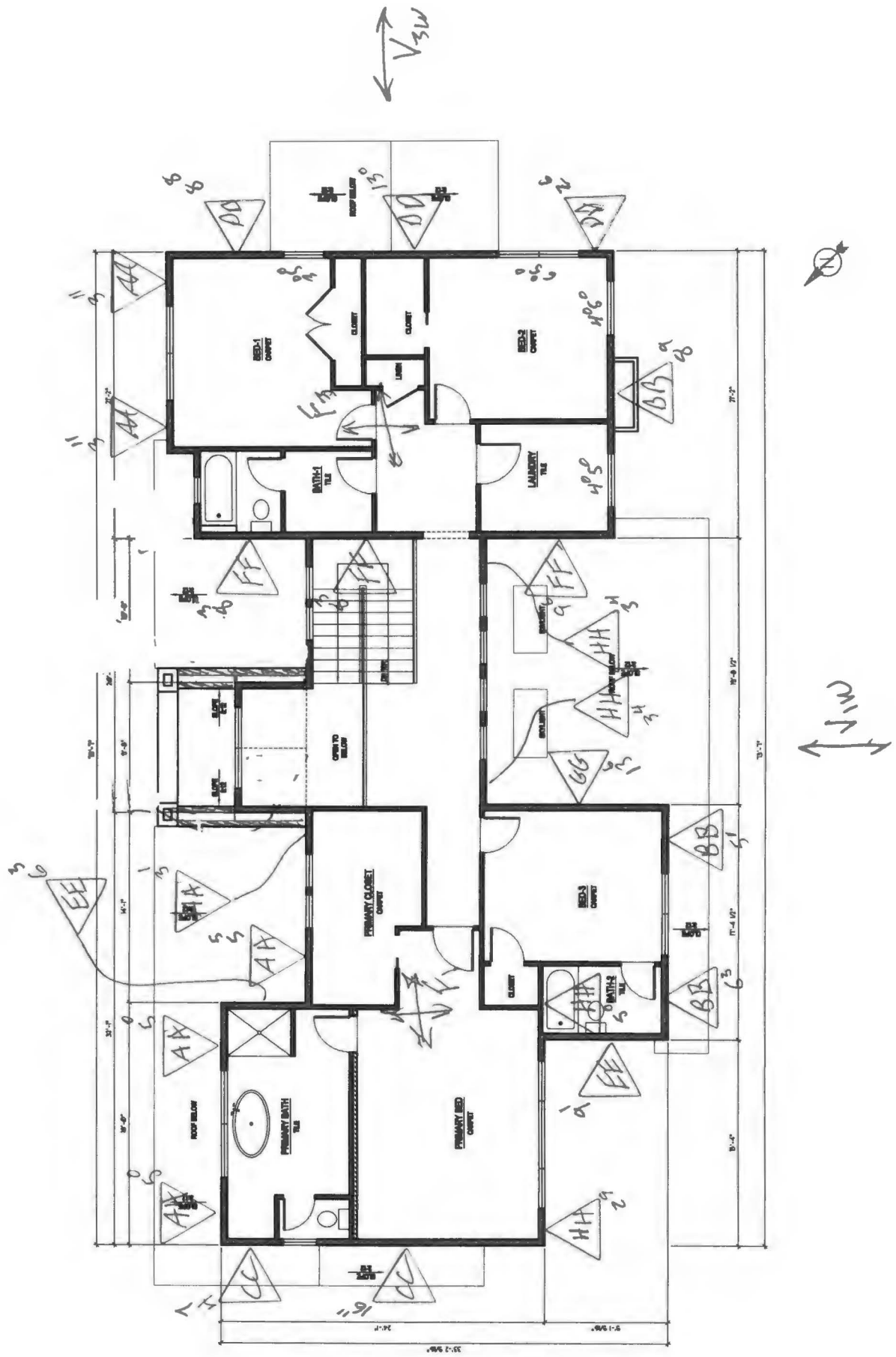
$$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 0 \cdot ft^2 + (p_{ww2} - p_{lw1}) \cdot 430 \cdot ft^2 = 14738.06 \text{ lb}$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 150 \cdot ft^2 + (p_{ww1} - p_{lw2}) \cdot 390 \cdot ft^2 = 15606.06 \text{ lb}$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0 \cdot ft^2 + (p_{ww2} - p_{lw2}) \cdot 590 \cdot ft^2 = 16736.67 \text{ lb}$$



WALL AA:

Story Shear due to Wind: $V_{3W} = 15606.06 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 34 \cdot \text{ft}$

Distance between shear walls: $L_2 = 24 \cdot \text{ft}$

Shear Wall Length:

$$L_{aaw} := \left(2 \cdot 5 + 5.42 + 3.08 \left(1.25 - 0.125 \left(\frac{9}{3.08} \right) \right) + 2 \cdot 3.92 \left(1.25 - 0.125 \left(\frac{9}{3.92} \right) \right) \right) \cdot \text{ft} = 25.70 \text{ ft}$$

$$L_{aas} := \left(2 \cdot 3.92 \left(1.25 - 0.125 \left(\frac{9}{3.92} \right) \right) \right) \cdot \text{ft} = 7.55 \text{ ft} \quad \text{Greatest Seismic PLF Case}$$

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore $C_o = 1.00$
 per AF&PA SDPWS Table 4.3.3.5

Wind Force:
$$v_{aa} := \frac{0.6 \cdot V_{3W} \cdot L_1}{L_2 \cdot 2} \cdot \frac{1}{L_{aaw}}$$

Seismic Force:
$$E_{aa} := \frac{0.7 \cdot (F_1) \cdot L_1}{L_2 \cdot 2} \cdot \frac{1}{L_{aas}}$$

$$v_{aa} = 128.62 \frac{\text{lb}}{\text{ft}}$$

$$\frac{v_{aa}}{C_o} = 128.62 \frac{\text{lb}}{\text{ft}}$$

$$E_{aa} = 215.91 \frac{\text{lb}}{\text{ft}}$$

$$\frac{E_{aa}}{C_o} = 215.91 \frac{\text{lb}}{\text{ft}}$$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning:

$$L_{aa} = 3.92 \cdot \text{ft}$$

Plate Height: $Pt = 9 \cdot \text{ft}$

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

$$DLR_{aa} := \frac{W_{aa} \cdot L_{aa}}{2} = 235.20 \text{ lb}$$

Chord Force:

$$CF_{aa_w} := \frac{v_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 1157.55 \text{ lb}$$

$$CF_{aa_s} := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 1943.22 \text{ lb}$$

Holdown Force:

$$HDF_{aa_w} := CF_{aa_w} - 0.6 \cdot DLR_{aa} = 1016.43 \text{ lb}$$

$$HDF_{aa_s} := CF_{aa_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{aa} = 1833.28 \text{ lb}$$

Simpson MSTC40 at Wall or dropped beam & MSTC28 at Rim Beam

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

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

$$Z_B = 860 \cdot \text{lb} \quad C_D = 1.6$$

$$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{aa}} = 1.27 \text{ ft} \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 0.76 \text{ ft}$$

$$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{aa}} = 10.70 \text{ ft} \quad \frac{(Z_B \cdot C_D \cdot C_o)}{E_{aa}} = 6.37 \text{ ft}$$

16d Sinkers @ 8" o.c.

5/8" dia. anchors @ 72" o.c.

WALL BB:

Story Shear due to Wind: $V_{3W} = 15606.06 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 34 \cdot \text{ft}$

Distance between shear walls: $L_j = 10 \cdot \text{ft}$

Shear Wall Length:

$L_{bbw} = (6.25 + 5.08 + 8.75) \cdot \text{ft} = 20.08 \text{ ft}$

$L_{bbs} = (8.75) \cdot \text{ft} = 8.75 \text{ ft}$ Greatest Seismic PLF Case

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}}\right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_d = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_{bb} = \frac{0.6 \cdot V_{3W} \cdot L_j}{L_1 \cdot 2} \cdot L_{bbw}$

Seismic Force: $E_{bb} = \frac{0.7 \cdot F_3 \cdot L_j}{L_1 \cdot 2} \cdot L_{bbs}$

$v_{bb} = 68.58 \frac{\text{lb}}{\text{ft}}$ $\frac{v_{bb}}{C_o} = 68.58 \frac{\text{lb}}{\text{ft}}$

$E_{bb} = 77.63 \frac{\text{lb}}{\text{ft}}$ $\frac{E_{bb}}{C_o} = 77.63 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overtuming: $L_{bb} = 5.08 \cdot \text{ft}$

Plate Height: $P_t = 9 \cdot \text{ft}$

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

$DLR_{bb} = \frac{W_{bb} \cdot L_{bb}}{2} = 304.80 \text{ lb}$

Chord Force:

$CF_{bb_w} = \frac{v_{bb} \cdot L_{bb} \cdot P_t}{C_o \cdot L_{bb}} = 617.18 \text{ lb}$

$CF_{bb_s} = \frac{E_{bb} \cdot L_{bb} \cdot P_t}{C_o \cdot L_{bb}} = 698.64 \text{ lb}$

Holdown Force:

$HDF_{bb_w} = CF_{bb_w} - 0.6 \cdot DLR_{bb} = 434.30 \text{ lb}$

$HDF_{bb_s} = CF_{bb_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{bb} = 556.15 \text{ lb}$

No Holdown Required

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

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{bb}} = 2.38 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 2.10 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{bb}} = 20.07 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{bb}} = 17.73 \text{ ft}$

16d Sinkers @ 16" o.c.

5/8" dia. anchors @ 72" o.c.

WALL CC:

Story Shear due to Wind: $V_{1W} = 13481.93 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 74 \text{ ft}$

Distance between shear walls: $L_2 = 18 \text{ ft}$

Shear Wall Length: $L_{cc} = (16.92 + 4.58) \cdot \text{ft} = 21.50 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}}\right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_s = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_{cc} = \frac{0.6 \cdot V_{1W} \cdot L_1}{L_t \cdot 2} \cdot \frac{L_2}{L_{cc}}$

Seismic Force: $E_{cc} = \frac{\rho \cdot (F_1 + F_3) \cdot L_1}{L_2 \cdot 2} \cdot \frac{L_2}{L_{cc}}$

$v_{cc} = 45.76 \frac{\text{lb}}{\text{ft}}$ $\frac{v_{cc}}{C_o} = 45.76 \frac{\text{lb}}{\text{ft}}$

$E_{cc} = 57.15 \frac{\text{lb}}{\text{ft}}$ $\frac{E_{cc}}{C_o} = 57.15 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{cc} = 4.58 \cdot \text{ft}$

Plate Height: $P_t = 9 \cdot \text{ft}$

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

$DLR_{cc} = \frac{W_{cc} \cdot L_{cc}}{2} = 549.60 \text{ lb}$

Chord Force:

$CF_{cc_w} = \frac{v_{cc} \cdot L_{cc} \cdot P_t}{C_o \cdot L_{cc}} = 411.83 \text{ lb}$

$CF_{cc_s} = \frac{E_{cc} \cdot L_{cc} \cdot P_t}{C_o \cdot L_{cc}} = 514.36 \text{ lb}$

Holdown Force:

$HDF_{cc_w} = CF_{cc_w} - 0.6 \cdot DLR_{cc} = 82.07 \text{ lb}$

$HDF_{cc_s} = CF_{cc_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{cc} = 257.44 \text{ lb}$

No Holdown Required

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

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{cc}} = 3.57 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 2.86 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{cc}} = 30.07 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{cc}} = 24.08 \text{ ft}$

16d Sinkers @ 16" o.c.

5/8" dia. anchors @ 72" o.c.

WALL DD:

Story Shear due to Wind: $V_{1W} = 13481.93 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 74 \cdot \text{ft}$

Distance between shear walls: $L_d = 21 \cdot \text{ft}$

Shear Wall Length: $L_{dd} = \left(8.67 + 13 + 2.5 \left(1.25 - 0.125 \left(\frac{9}{2.5} \right) \right) \right) \cdot \text{ft} = 23.67 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_D = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_{dd} = \frac{0.6 \cdot V_{1W} \cdot L_1}{L_d \cdot 2}$

Seismic Force: $E_{dd} = \frac{\rho \cdot (F_1 + F_3) \cdot L_1}{L_d \cdot 2}$

$v_{dd} = 48.49 \frac{\text{lb}}{\text{ft}}$

$\frac{v_{dd}}{C_o} = 48.49 \frac{\text{lb}}{\text{ft}}$

$E_{dd} = 60.56 \frac{\text{lb}}{\text{ft}}$

$\frac{E_{dd}}{C_o} = 60.56 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 280 plf

Dead Load Resisting Overturning: $L_{dd} = 2.5 \cdot \text{ft}$

Plate Height: $P_t = 9 \cdot \text{ft}$

$W_{dd} = (15 \cdot \text{psf}) \cdot 11 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLR_{dd} = \frac{W_{dd} \cdot L_{dd}}{2} = 318.75 \text{ lb}$

Chord Force:

$CF_{dd_w} = \frac{v_{dd} \cdot L_{dd} \cdot P_t}{C_o \cdot L_{dd}} = 436.42 \text{ lb}$

$CF_{dd_s} = \frac{E_{dd} \cdot L_{dd} \cdot P_t}{C_o \cdot L_{dd}} = 545.07 \text{ lb}$

Holdown Force:

$HDF_{dd_w} = CF_{dd_w} - 0.6 \cdot DLR_{dd} = 245.17 \text{ lb}$

$HDF_{dd_s} = CF_{dd_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{dd} = 396.07 \text{ lb}$

No Holdown Required

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

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{dd}} = 3.37 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 2.69 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{dd}} = 28.38 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{dd}} = 22.72 \text{ ft}$

16d Sinkers @ 16" o.c.

5/8" dia. anchors @ 72" o.c.

WALLEE:

Story Shear due to Wind: $V_{1W} = 13481.93 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 74 \text{ ft}$

Distance between shear walls: $L_2 = 18 \text{ ft}$ $L_3 = 14 \text{ ft}$

Shear Wall Length: $L_{ee} = (6.25 + 9) \cdot \text{ft} = 15.25 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}}\right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_o = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force:
$$v_{ee} = \frac{0.6 \cdot V_{1W} \cdot \frac{L_1 + L_2}{2}}{L_1 \cdot L_{ee}}$$

Seismic Force: $C_s = 1.3$
$$E_{ee} = \frac{\rho \cdot \frac{0.7 \cdot (F_1 + F_3) \cdot \frac{L_1 + L_2}{2}}{L_1}}{L_{ee}}$$

$v_{ee} = 114.69 \frac{\text{lb}}{\text{ft}}$ $\frac{v_{ee}}{C_o} = 114.69 \frac{\text{lb}}{\text{ft}}$

$E_{ee} = 143.24 \frac{\text{lb}}{\text{ft}}$ $\frac{E_{ee}}{C_o} = 143.24 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{ow} = 6.25 \cdot \text{ft}$

Plate Height: $P_1 = 9 \cdot \text{ft}$

$W_{ow} = (15 \cdot \text{psf}) \cdot 10 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_1 + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DL_{Ree} = \frac{W_{ow} \cdot L_{ee}}{2} = 750.00 \text{ lb}$

Chord Force:

$CF_{ee_w} = \frac{v_{ee} \cdot L_{ee} \cdot P_1}{C_o \cdot L_{ee}} = 1032.20 \text{ lb}$

$CF_{ee_s} = \frac{E_{ee} \cdot L_{ee} \cdot P_1}{C_o \cdot L_{ee}} = 1289.18 \text{ lb}$

Holdown Force:

$HD_{Fee_w} = CF_{ee_w} - 0.6 \cdot DL_{Ree} = 582.20 \text{ lb}$

$HD_{Fee_s} = CF_{ee_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{Ree} = 938.58 \text{ lb}$

Simpson MSTC28 strap

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{ee}} = 1.42 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ee}} = 1.14 \text{ ft}$

16d Sinker @ 12" o.c.

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{ee}} = 12.00 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ee}} = 9.61 \text{ ft}$

5/8" dia. anchors @ 72" o.c.

WALL FF:

Story Shear due to Wind: $V_{IW} = 13481.93 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 74 \cdot \text{ft}$

Distance between shear walls: $L_2 = 21 \cdot \text{ft}$ $L_3 = 20 \cdot \text{ft}$

Shear Wall Length: $L_{ff} = (9.5 + 2 \cdot 8.25) \cdot \text{ft} = 26.00 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}}\right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_2 = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force:
$$v_{ff} = \frac{0.6 \cdot V_{IW} \cdot L_1 + L_2}{L_1 \cdot L_{ff}}$$

Seismic Force: $C_1 = 1.3$
$$E_{ff} = \frac{\rho \cdot 0.7 \cdot (F_1 + F_3) \cdot L_1 + L_2}{L_1 \cdot L_{ff}}$$

$v_{ff} = 86.19 \frac{\text{lb}}{\text{ft}}$

$\frac{v_{ff}}{C_o} = 86.19 \frac{\text{lb}}{\text{ft}}$

$E_{ff} = 107.65 \frac{\text{lb}}{\text{ft}}$

$\frac{E_{ff}}{C_o} = 107.65 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{ff} = 8.25 \cdot \text{ft}$

Plate Height: $P_1 = 9 \cdot \text{ft}$

$W_{ff} = (15 \cdot \text{psf}) \cdot 11 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_1 + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLR_{ff} = \frac{W_{ff} \cdot L_{ff}}{2} = 1051.88 \text{ lb}$

Chord Force:

$CF_{ff_w} = \frac{v_{ff} \cdot L_{ff} \cdot P_1}{C_o \cdot L_{ff}} = 775.70 \text{ lb}$

$CF_{ff_s} = \frac{E_{ff} \cdot L_{ff} \cdot P_1}{C_o \cdot L_{ff}} = 968.82 \text{ lb}$

Holdown Force:

$HDF_{ff_w} = CF_{ff_w} - 0.6 \cdot DLR_{ff} = 144.58 \text{ lb}$

$HDF_{ff_s} = CF_{ff_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{ff} = 477.11 \text{ lb}$

No Holdown Required

Base Plate Nail Spacing (2018 NDS Table 12N)

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{ff}} = 1.89 \text{ ft}$

$\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ff}} = 1.52 \text{ ft}$

16d Sinkers @ 16" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{ff}} = 15.96 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ff}} = 12.78 \text{ ft}$

5/8" dia. anchors @ 72" o.c.

WALL GG:

Story Shear due to Wind: $V_{IW} = 13481.93 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 74 \text{ ft}$

Distance between shear walls: $L_1 = 14 \text{ ft}$ $L_2 = 20 \text{ ft}$

Shear Wall Length: $L_{gg} = (13.5) \cdot \text{ft} = 13.50 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \text{ ft}}{10 \text{ ft}}\right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_o = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_{gg} := \frac{0.6 \cdot V_{IW} \cdot \frac{L_1 + L_2}{2}}{L_1 \cdot L_{gg}}$

Seismic Force: $\rho = 1.3$ $E_{gg} := \frac{\rho \cdot \frac{0.7 \cdot (F_1 + F_3) \cdot (L_1 + L_2)}{2}}{L_1 \cdot L_{gg}}$

$v_{gg} = 137.65 \frac{\text{lb}}{\text{ft}}$ $\frac{v_{gg}}{C_o} = 137.65 \frac{\text{lb}}{\text{ft}}$

$E_{gg} = 171.92 \frac{\text{lb}}{\text{ft}}$ $\frac{E_{gg}}{C_o} = 171.92 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 8" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{gg} = 13.5 \text{ ft}$

Plate Height: $H = 9 \text{ ft}$

$W_{gg} := (15 \cdot \text{psf}) \cdot 9 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLR_{gg} := \frac{W_{gg} \cdot L_{gg}}{2} = 1518.75 \text{ lb}$

Chord Force:

$CF_{gg_w} := \frac{v_{gg} \cdot L_{gg} \cdot Pt}{C_o \cdot L_{gg}} = 1238.88 \text{ lb}$

$CF_{gg_s} := \frac{E_{gg} \cdot L_{gg} \cdot Pt}{C_o \cdot L_{gg}} = 1547.32 \text{ lb}$

Holdown Force:

$HDF_{gg_w} := CF_{gg_w} - 0.6 \cdot DLR_{gg} = 327.63 \text{ lb}$

$HDF_{gg_s} := CF_{gg_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{gg} = 837.35 \text{ lb}$

Simpson MSTC28 strap

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{gg}} = 1.19 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{gg}} = 0.95 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{gg}} = 10.00 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{gg}} = 8.00 \text{ ft}$

16d Sinkers @ 8" o.c.

5/8" dia. anchors @ 72" o.c.

WALL HH:

Story Shear due to Wind: $V_{3W} = 15606.06 \text{ lb}$

Story Shear due to Seismic: $F_1 = 6026.71 \text{ lb}$ $F_3 = 5075.56 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 34 \text{ ft}$

Distance between shear walls: $L_2 = 24 \text{ ft}$ $L_3 = 10 \text{ ft}$

Shear Wall Length: $L_{hh} = \left(2.75 \left(1.25 - 0.125 \left(\frac{9}{2.75} \right) \right) + 5 + 2 \cdot 3.33 \left(1.25 - 0.125 \left(\frac{9}{3.33} \right) \right) \right) \cdot \text{ft} = 13.39 \text{ ft}$

Percent full height sheathing: $\left(\frac{10 \text{ ft}}{10 \text{ ft}} \right) \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_d = 1.00$ per AF&PA SDPWS Table 4.3.3.5

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

Seismic Force: $C_s = 1.3$ $E_{hh} = \frac{p \cdot \frac{0.7 \cdot (F_1 + F_3) \cdot \frac{L_1 + L_2}{2}}{L_1}}{L_{hh}}$

$v_{hh} = 349.72 \frac{\text{lb}}{\text{ft}}$

$\frac{v_{hh}}{C_o} = 349.72 \frac{\text{lb}}{\text{ft}}$

$E_{hh} = 377.33 \frac{\text{lb}}{\text{ft}}$

$\frac{E_{hh}}{C_o} = 377.33 \frac{\text{lb}}{\text{ft}}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
 Wind Capacity = 532 plf
 Seismic Capacity = 380 plf

Dead Load Resisting Overturning: $L_{hh} = 2.75 \text{ ft}$

Plate Height: $P_t = 9 \text{ ft}$

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

$DLR_{hh} = \frac{W_{hh} \cdot L_{hh}}{2} = 165.00 \text{ lb}$

Chord Force:

$CF_{hh_w} = \frac{v_{hh} \cdot L_{hh} \cdot P_t}{C_o \cdot L_{hh}} = 3147.44 \text{ lb}$

$CF_{hh_s} = \frac{E_{hh} \cdot L_{hh} \cdot P_t}{C_o \cdot L_{hh}} = 3395.99 \text{ lb}$

Holdown Force:

$HDF_{hh_w} = CF_{hh_w} - 0.6 \cdot DLR_{hh} = 3048.44 \text{ lb}$

$HDF_{hh_s} = CF_{hh_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_{hh} = 3318.85 \text{ lb}$

Simpson MST48 to wall or dropped beam or MSTC28 strap to Rim/Flush beam

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_{hh}} = 0.47 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{hh}} = 0.43 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_{hh}} = 3.93 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{hh}} = 3.65 \text{ ft}$

16d Sinkers @ 4" o.c.

5/8" dia. anchors @ 42" o.c.

WALL A:

Story Shear due to Wind: $V_{AW} = 16736.67 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 33 \cdot \text{ft}$

Distance between shear walls: $L_2 = 33 \cdot \text{ft}$

Shear Wall Length: $L_a = \left(2 \cdot 2 + 2 \cdot 2.75 \left(1.25 - 0.125 \left(\frac{8}{2.75} \right) \right) + 4.17 + 6.17 + 7 + 2 \cdot 3.08 \left(1.25 - 0.125 \left(\frac{10}{3.08} \right) \right) \right) \cdot \text{ft} = 31.42 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_2 = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_a = \frac{v_{aw} \cdot L_{aw} + \left(\frac{0.6 \cdot V_{AW} \cdot L_1}{L_1 \cdot 2} \right)}{L_a}$

Seismic Force: $E_a = \frac{E_{aw} \cdot L_{aw} + \left(\rho \cdot \frac{0.7 \cdot (F_2 + F_4) \cdot L_1}{L_1 \cdot 2} \right)}{L_a}$

$v_a = 265.03 \frac{\text{lb}}{\text{ft}}$

$\frac{v_a}{C_o} = 265.03 \frac{\text{lb}}{\text{ft}}$

$E_a = 288.06 \frac{\text{lb}}{\text{ft}}$

$\frac{E_a}{C_o} = 288.06 \frac{\text{lb}}{\text{ft}}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
 Wind Capacity = 532 plf
 Seismic Capacity = 380 plf

Restraint Panel Height = 10ft Maximum
 Restraint Panel Width = 2ft-0in Minimum
 Allowable Shear per Panel = 1125 lbs Seismic & 1575 lbs Wind

See APF Technical Topic TT-100
 "A Portal Frame with Hold Downs for
 Engineered Applications" (Emphasis Added)

Shear per Panel: $V_{s1} = (2 \cdot \text{ft} \cdot E_a) = 576.12 \text{ lb}$ O.K.

$V_{s2} = (2 \cdot \text{ft} \cdot v_a) = 530.05 \text{ lb}$ O.K.

Dead Load Resisting Overturning: $L_a = 2.75 \cdot \text{ft}$

Plate Height: $P_1 = 8 \cdot \text{ft}$

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

$DLRa = \frac{W_a \cdot L_a}{2} = 171.88 \text{ lb}$

Chord Force:

$CFa_w = \frac{v_a \cdot L_a \cdot P_t}{C_o \cdot L_a} = 2120.21 \text{ lb}$

$CFa_s = \frac{E_a \cdot L_a \cdot P_t}{C_o \cdot L_a} = 2304.47 \text{ lb}$

$CFa_w + CFaa_w = 3277.77 \text{ lb}$

$CFa_s + CFaa_s = 4247.69 \text{ lb}$

Holddown Force:

$HDFa_w = CFa_w - 0.6 \cdot DLRa = 2017.09 \text{ lb}$

$HDFa_s = CFa_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRa = 2224.12 \text{ lb}$

$HDFa_w + HDFaa_w = 3033.52 \text{ lb}$

$HDFa_s + HDFaa_s = 4057.40 \text{ lb}$

Simpson STHD10/RJ

Simpson HDU4 w/ SB5/8x24 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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_a} = 0.62 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.57 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_a} = 5.19 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_a} = 4.78 \text{ ft}$

16d Sinkers @ 6" o.c.

5/8" dia. anchors @ 60" o.c.

WALL B:

Story Shear due to Wind: $V_{1W} = 16736.67 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 33 \cdot \text{ft}$

Distance between shear walls: $L_2 = 33 \cdot \text{ft}$

Shear Wall Length: $L_b = (21.5 + 3.58 + 8.25) \cdot \text{ft} = 33.33 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_d = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force:

Seismic Force:

$$v_b = \frac{(v_{bb} \cdot L_{bbw}) + (v_{hh} \cdot L_{hh}) + \left(\frac{0.6 \cdot V_{1W} \cdot L_1}{L_1 \cdot 2} \right)}{L_b}$$

$$E_b = 1.3 \frac{(E_{bb} \cdot L_{bbw}) + (E_{hh} \cdot L_{hh}) + \left(\frac{0.7 \cdot (F_2 + F_4) \cdot L_1}{L_1 \cdot 2} \right)}{L_b}$$

$$v_b = 332.43 \frac{\text{lb}}{\text{ft}}$$

$$\frac{v_b}{C_o} = 332.43 \frac{\text{lb}}{\text{ft}}$$

$$E_b = 303.38 \frac{\text{lb}}{\text{ft}}$$

$$\frac{E_b}{C_o} = 303.38 \frac{\text{lb}}{\text{ft}}$$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
 Wind Capacity = 532 plf
 Seismic Capacity = 380 plf

Dead Load Resisting Overturning: $L_b = 3.58 \cdot \text{ft}$

Plate Height: $H = 10 \cdot \text{ft}$

$$W_p = (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10 \cdot \text{psf}) \cdot 1 \cdot \text{ft}$$

$$DLR_b = \frac{W_b \cdot L_b}{2} = 196.90 \text{ lb}$$

Chord Force:

$$CF_{b_w} = \frac{v_b \cdot L_b \cdot Pt}{C_o \cdot L_b} = 3324.28 \text{ lb}$$

$$CF_{b_s} = \frac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b} = 3033.82 \text{ lb}$$

$$CF_{b_w} + CF_{bb_w} = 3941.46 \text{ lb}$$

$$CF_{b_s} + CF_{bb_s} = 3732.45 \text{ lb}$$

Holddown Force:

$$HDF_{b_w} = CF_{b_w} - 0.6 \cdot DLR_b = 3206.14 \text{ lb}$$

$$HDF_{b_s} = CF_{b_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_b = 2941.77 \text{ lb}$$

$$HDF_{b_w} + HDF_{bb_w} = 3640.44 \text{ lb}$$

$$HDF_{b_s} + HDF_{bb_s} = 3497.92 \text{ lb}$$

Simpson STHD14/RJ

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

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

$$Z_B = 860 \cdot \text{lb} \quad C_D = 1.6$$

$$\frac{(Z_N \cdot C_D \cdot C_o)}{v_b} = 0.49 \text{ ft} \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.54 \text{ ft}$$

$$\frac{(Z_B \cdot C_D \cdot C_o)}{v_b} = 4.14 \text{ ft} \quad \frac{(Z_B \cdot C_D \cdot C_o)}{E_b} = 4.54 \text{ ft}$$

16d Sinkers @ 6" o.c.

5/8" dia. anchors @ 48" o.c.

WALL C:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \cdot \text{ft}$

Distance between shear walls: $L_2 = 23 \cdot \text{ft}$

Shear Wall Length: $L_c = (2 \cdot 5.5 + 14.08) \cdot \text{ft} = 25.08 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_d = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force:
$$v_c = \frac{v_{cc} \cdot L_{cc} + \left(\frac{0.6 \cdot V_{2W} \cdot L_1}{L_1 \cdot 2} \right)}{L_c}$$

Seismic Force: $\rho = 1.3$
$$E_c = \frac{E_{cc} \cdot L_{cc} + \left(\rho \cdot \frac{0.7 \cdot (F_2 + F_4) \cdot L_1}{L_1 \cdot 2} \right)}{L_c}$$

$v_c = 87.50 \frac{\text{lb}}{\text{ft}}$

$\frac{v_c}{C_o} = 87.50 \frac{\text{lb}}{\text{ft}}$

$E_c = 87.22 \frac{\text{lb}}{\text{ft}}$

$\frac{E_c}{C_o} = 87.22 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_c = 5.5 \cdot \text{ft}$

Plate Height: $P_t = 10 \cdot \text{ft}$

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

$DLR_c = \frac{W_c \cdot L_c}{2} = 357.50 \text{ lb}$

Chord Force:

$CF_{c_w} = \frac{v_c \cdot L_c \cdot P_t}{C_o \cdot L_c} = 874.98 \text{ lb}$

$CF_{c_s} = \frac{E_c \cdot L_c \cdot P_t}{C_o \cdot L_c} = 872.20 \text{ lb}$

$CF_{c_w} + CF_{c_s} = 1286.81 \text{ lb}$

$CF_{c_s} + CF_{c_s} = 1386.56 \text{ lb}$

Holdown Force:

$HDF_{c_w} = CF_{c_w} - 0.6 \cdot DLR_c = 660.48 \text{ lb}$

$HDF_{c_s} = CF_{c_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_c = 705.08 \text{ lb}$

$HDF_{c_w} + HDF_{c_s} = 742.55 \text{ lb}$

$HDF_{c_s} + HDF_{c_s} = 962.52 \text{ lb}$

No Holdown Required

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

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_c} = 1.87 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 1.87 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_c} = 15.73 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_c} = 15.78 \text{ ft}$

16d Sinkers @ 16" o.c.

5/8" dia. anchors @ 72" o.c.

WALL D:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \text{ ft}$

Distance between shear walls: $L_2 = 21 \text{ ft}$ $L_3 = 8 \text{ ft}$

Shear Wall Length: $L_d = \left(8.67 + 2.5 \left(1.25 - 0.125 \left(\frac{5.5}{2.5} \right) \right) \right) \cdot \text{ft} = 11.11 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 0ft-0in, Therefore
 $C_o = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vd = \frac{vdd \cdot Ldd + \left(\frac{0.6 \cdot V_{2W} \cdot L_1 + L_2}{L_1} \cdot \frac{L_1 + L_2}{2} \right)}{L_d}$

Seismic Force: $E_d = \frac{E_{dd} \cdot Ldd + \left(p \cdot \frac{0.7 \cdot (F_2 + F_4) \cdot L_1 + L_2}{L_1} \cdot \frac{L_1 + L_2}{2} \right)}{L_d}$

$vd = 240.76 \frac{\text{lb}}{\text{ft}}$

$\frac{vd}{C_o} = 240.76 \frac{\text{lb}}{\text{ft}}$

$E_d = 237.89 \frac{\text{lb}}{\text{ft}}$

$\frac{E_d}{C_o} = 237.89 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_d = 2.5 \cdot \text{ft}$

Plate Height: $P_t = 5.5 \cdot \text{ft}$

$W_d = (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 10 \cdot \text{ft}$

$DLRd = \frac{W_d \cdot L_d}{2} = 193.75 \text{ lb}$

Chord Force:

$CFd_w = \frac{vd \cdot L_d \cdot P_t}{C_o \cdot L_d} = 1324.17 \text{ lb}$

$CFd_s = \frac{E_d \cdot L_d \cdot P_t}{C_o \cdot L_d} = 1308.40 \text{ lb}$

$CFd_w + CFdd_w = 1760.59 \text{ lb}$

$CFd_s + CFdd_s = 1853.47 \text{ lb}$

Holdown Force:

$HDFd_w = CFd_w - 0.6 \cdot DLRd = 1207.92 \text{ lb}$

$HDFd_s = CFd_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRd = 1217.83 \text{ lb}$

$HDFd_w + HDFdd_w = 1453.09 \text{ lb}$

$HDFd_s + HDFdd_s = 1613.90 \text{ lb}$

Simpson STHD10/RJ

Base Plate Nail Spacing (2018 NDS Table 12N)

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vd} = 0.68 \text{ ft}$

$\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 0.69 \text{ ft}$

16d Sinker @ 8" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vd} = 5.72 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{E_d} = 5.78 \text{ ft}$

5/8" dia. anchors @ 60" o.c.

WALLE:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \cdot \text{ft}$

Distance between shear walls: $L_1 = 23 \cdot \text{ft}$ $L_2 = 12 \cdot \text{ft}$

Shear Wall Length: $L_e = (5 + 10.58) \cdot \text{ft} = 15.58 \text{ ft}$

Percent full height sheathing: $\frac{15.58 \cdot \text{ft}}{16.08 \cdot \text{ft}} \cdot 100 = 96.89$

Max Opening Height = 10ft-0in, Therefore
 $C_o = 0.95$ per AF&PA SDPWS Table 4.3.3.5

Wind Force:
$$v_e = \frac{v_{ee} \cdot L_{ee} + \left(\frac{0.6 \cdot V_{2W} \cdot L_2 + L_2}{L_1} \right)}{L_e}$$

Seismic Force: $\mu = 1.3$
$$E_e = \frac{E_{ee} \cdot L_{ee} + \left(\mu \cdot \frac{0.7 \cdot (F_2 + F_4) \cdot L_2 + L_2}{L_1} \right)}{L_e}$$

$v_e = 230.50 \frac{\text{lb}}{\text{ft}}$

$\frac{v_e}{C_o} = 242.64 \frac{\text{lb}}{\text{ft}}$

$E_e = 233.85 \frac{\text{lb}}{\text{ft}}$

$\frac{E_e}{C_o} = 246.16 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_e = 15.58 \cdot \text{ft}$

Plate Height: $H = 10 \cdot \text{ft}$

$W_e = (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot H + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLRe = \frac{W_e \cdot L_e}{2} = 779.00 \text{ lb}$

Chord Force:

$CF_{e_w} = \frac{v_e \cdot L_e \cdot Pt}{C_o \cdot L_e} = 2426.37 \text{ lb}$

$CF_{e_s} = \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e} = 2461.58 \text{ lb}$

$CF_{e_w} + CF_{e_s} = 3458.57 \text{ lb}$

$CF_{e_s} + CF_{e_s} = 3750.76 \text{ lb}$

Holddown Force:

$HDF_{e_w} = CF_{e_w} - 0.6 \cdot DLRe = 1958.97 \text{ lb}$

$HDF_{e_s} = CF_{e_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRe = 2097.42 \text{ lb}$

$HDF_{e_w} + HDF_{e_s} = 2541.17 \text{ lb}$

$HDF_{e_s} + HDF_{e_s} = 3036.00 \text{ lb}$

Simpson STHD14/RJ or 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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_e} = 0.67 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 0.66 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_e} = 5.67 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_e} = 5.59 \text{ ft}$

16d Sinkers @ 8" o.c.

5/8" dia. anchors @ 60" o.c.

WALL E:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \cdot \text{ft}$

Distance between shear walls: $L_1 = 20 \cdot \text{ft}$ $L_2 = 21 \cdot \text{ft}$

Shear Wall Length: $L_f = \left(5 + 10.42 + 2.5 \left(1.25 - 0.125 \left(\frac{8}{2.5} \right) \right) \right) \cdot \text{ft} = 17.55 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 10ft-0in, Therefore
 $C_D = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_f = \frac{v_{ff} \cdot L_{ff} + \left(\frac{0.6 \cdot V_{2W} \cdot L_1 + L_2}{L_1} \right)}{L_f}$

Seismic Force: $E_f = 1.3 \frac{E_{ff} \cdot L_{ff} + \left(\rho \cdot \frac{0.7 \cdot (F_2 + F_4) \cdot L_1 + L_2}{L_1} \right)}{L_f}$

$v_f = 250.73 \frac{\text{lb}}{\text{ft}}$

$\frac{v_f}{C_o} = 250.73 \frac{\text{lb}}{\text{ft}}$

$E_f = 256.93 \frac{\text{lb}}{\text{ft}}$

$\frac{E_f}{C_o} = 256.93 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 364 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{r1} = 2.5 \cdot \text{ft}$

Plate Height: $P_H = 8 \cdot \text{ft}$

$W_{r1} = (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot P_t + (10 \cdot \text{psf}) \cdot 0 \cdot \text{ft}$

$DLR_f = \frac{W_{r1} \cdot L_f}{2} = 100.00 \text{ lb}$

Chord Force:

$CF_{f_w} = \frac{v_f \cdot L_f \cdot P_t}{C_o \cdot L_f} = 2005.81 \text{ lb}$

$CF_{f_s} = \frac{E_f \cdot L_f \cdot P_t}{C_o \cdot L_f} = 2055.45 \text{ lb}$

$CF_{f_w} + CF_{f_s} = 2781.51 \text{ lb}$

$CF_{f_s} + CF_{f_s} = 3024.27 \text{ lb}$

Holddown Force:

$HDF_{f_w} = CF_{f_w} - 0.6 \cdot DLR_f = 1945.81 \text{ lb}$

$HDF_{f_s} = CF_{f_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_f = 2008.70 \text{ lb}$

$HDF_{f_w} + HDF_{f_s} = 2090.39 \text{ lb}$

$HDF_{f_s} + HDF_{f_s} = 2485.81 \text{ lb}$

Simpson MSTC40 at framed wall & STHD10/RJ or HDU2 w/ SSTB16 or PAB5 anchor at foundation

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_f} = 0.65 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 0.64 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_f} = 5.49 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_f} = 5.36 \text{ ft}$

16d Sinkers @ 6" o.c.

5/8" dia. anchors @ 48" o.c.

WALL G:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \cdot \text{ft}$

Distance between shear walls: $L_2 = 12 \cdot \text{ft}$ $L_3 = 20 \cdot \text{ft}$

Shear Wall Length: $L_g = \left(4.33 \left(1.25 - 0.125 \left(\frac{10}{4.33} \right) \right) \right) \cdot \text{ft} = 4.16 \text{ ft}$

Percent full height sheathing: $\frac{10 \cdot \text{ft}}{10 \cdot \text{ft}} \cdot 100 = 100.00$

Max Opening Height = 10ft-0in, Therefore
 $C_1 = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $v_g = \frac{v_{gg} \cdot L_{gg} + \left(\frac{0.6 \cdot V_{2W}}{L_1} \cdot \frac{L_1 + L_2}{2} \right)}{L_g}$

Seismic Force: $E_g = \frac{E_{gg} \cdot L_{gg} + \left(\beta \cdot \frac{0.7 \cdot (F_2 + F_4)}{L_1} \cdot \frac{L_1 + L_2}{2} \right)}{L_g}$

$v_g = 851.09 \frac{\text{lb}}{\text{ft}}$

$\frac{v_g}{C_o} = 851.09 \frac{\text{lb}}{\text{ft}}$

$E_g = 878.04 \frac{\text{lb}}{\text{ft}}$

$\frac{E_g}{C_o} = 878.04 \frac{\text{lb}}{\text{ft}}$

P2-3: 7/16" Sheathing w/ 8d nails @ 3" O.C. Both Faces
 Wind Capacity = 1372 plf
 Seismic Capacity = 980 plf

Dead Load Resisting Overturning: $L_{2g} = 4.33 \cdot \text{ft}$

Plate Height: $Pt = 10 \cdot \text{ft}$

$W_g = (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10 \cdot \text{psf}) \cdot 6 \cdot \text{ft}$

$DLR_g = \frac{W_g \cdot L_g}{2} = 346.40 \text{ lb}$

Chord Force:

$CF_{g_w} = \frac{v_g \cdot L_g \cdot Pt}{C_o \cdot L_g} = 8510.92 \text{ lb}$

$CF_{g_s} = \frac{E_g \cdot L_g \cdot Pt}{C_o \cdot L_g} = 8780.44 \text{ lb}$

$CF_{g_w} + CF_{g_g} = 9749.80 \text{ lb}$

$CF_{g_s} + CF_{g_g} = 10327.75 \text{ lb}$

Holddown Force:

$HDF_{g_w} = CF_{g_w} - 0.6 \cdot DLR_g = 8303.08 \text{ lb}$

$HDF_{g_s} = CF_{g_s} - (0.6 - 0.14 \cdot S_{DS}) \cdot DLR_g = 8618.50 \text{ lb}$

$HDF_{g_w} + HDF_{g_g} = 8630.71 \text{ lb}$

$HDF_{g_s} + HDF_{g_g} = 9455.86 \text{ lb}$

Simpson HDU11 at 4x6 DF post minimum w/ PAB8 anchor embedded 9" into FOOTING and extended up to main floor wall with 1" all-thread rods & coupler nuts

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

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_N \cdot C_D \cdot C_o)}{v_g} = 0.19 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_g} = 0.19 \text{ ft}$

$\frac{(Z_B \cdot C_D \cdot C_o)}{v_g} = 1.62 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_g} = 1.57 \text{ ft}$

16d Sinkers @ 2" o.c. (Staggered)

5/8" dia. anchors @ 16" o.c.

WALL H:

Story Shear due to Wind: $V_{2W} = 14738.06 \text{ lb}$

Story Shear due to Seismic: $F_2 = 3784.40 \text{ lb}$ $F_4 = 3911.08 \text{ lb}$

Bldg Width in direction of Load: $L_1 = 84 \text{ ft}$

Distance between shear walls: $L_2 = 8 \text{ ft}$

Shear Wall Length: $L_h = \left(2 \cdot 3 \left(1.25 - 0.125 \left(\frac{10}{3} \right) \right) \right) \cdot \text{ft} = 5.00 \text{ ft}$

Percent full height sheathing: $\frac{10 \text{ ft}}{10 \text{ ft}} \cdot 100 = 100.00$

Max Opening Height = 10ft-0in, Therefore
 $C_d = 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vh = \frac{\left(\frac{0.6 \cdot V_{2W} \cdot L_1}{L_1 \cdot 2} \right)}{L_h}$

Seismic Force: $E_h = \frac{\left(\frac{0.7 \cdot (F_2 + F_4) \cdot L_1}{L_1 \cdot 2} \right)}{L_h}$

$vh = 84.22 \frac{\text{lb}}{\text{ft}}$ $\frac{vh}{C_o} = 84.22 \frac{\text{lb}}{\text{ft}}$

$E_h = 66.69 \frac{\text{lb}}{\text{ft}}$ $\frac{E_h}{C_o} = 66.69 \frac{\text{lb}}{\text{ft}}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
 Wind Capacity = 384 plf
 Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_h = 3 \text{ ft}$

Plate Height: $P_t = 10 \text{ ft}$

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

$DLRh = \frac{W_h \cdot L_h}{2} = 195.00 \text{ lb}$

Chord Force:

$CFh_w = \frac{vh \cdot L_h \cdot P_t}{C_o \cdot L_h} = 842.17 \text{ lb}$

$CFh_s = \frac{E_h \cdot L_h \cdot P_t}{C_o \cdot L_h} = 666.94 \text{ lb}$

Holdown Force:

$HDFh_w = CFh_w - 0.6 \cdot DLRh = 725.17 \text{ lb}$

$HDFh_s = CFh_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRh = 575.79 \text{ lb}$

No Holdown Required

Base Plate Nail Spacing (2018 NDS Table 12N)

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vh} = 1.94 \text{ ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_h} = 2.45 \text{ ft}$

16d Sinks @ 16" o.c.

Anchor Bolt Spacing (2018 NDS Table 12E)

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

$Z_B = 860 \cdot \text{lb}$ $C_D = 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vh} = 16.34 \text{ ft}$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_h} = 20.63 \text{ ft}$

5/8" dia. anchors @ 72" o.c.

Diaphragm Shear Check:

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

Unblocked Diaphragm Case 1 Wind Capacity = 322 plf & Seismic Capacity = 230 plf

Unblocked Diaphragm Case 2-6 Wind Capacity = 238 plf & Seismic Capacity = 170 plf

Wall Lines AA:

$$v_{aa} \cdot \frac{Laaw}{74 \cdot ft} = 44.66 \frac{lb}{ft}$$

$$E_{aa} \cdot \frac{Laas}{74 \cdot ft} = 22.03 \frac{lb}{ft}$$

Wall Lines EE:

$$v_{ee} \cdot \frac{Lee}{33 \cdot ft} = 53.00 \frac{lb}{ft}$$

$$E_{ee} \cdot \frac{Lee}{33 \cdot ft} = 66.20 \frac{lb}{ft}$$

Wall Lines BB:

$$v_{bb} \cdot \frac{Lbbw}{40 \cdot ft} = 34.43 \frac{lb}{ft}$$

$$E_{bb} \cdot \frac{Lbbs}{40 \cdot ft} = 16.98 \frac{lb}{ft}$$

Wall Lines FF:

$$v_{ff} \cdot \frac{Lff}{30 \cdot ft} = 74.70 \frac{lb}{ft}$$

$$E_{ff} \cdot \frac{Lff}{30 \cdot ft} = 93.29 \frac{lb}{ft}$$

Wall Lines CC:

$$v_{cc} \cdot \frac{Lcc}{24 \cdot ft} = 40.99 \frac{lb}{ft}$$

$$E_{cc} \cdot \frac{Lcc}{24 \cdot ft} = 51.20 \frac{lb}{ft}$$

Wall Lines GG:

$$v_{gg} \cdot \frac{Lgg}{33 \cdot ft} = 56.31 \frac{lb}{ft}$$

$$E_{gg} \cdot \frac{Lgg}{33 \cdot ft} = 70.33 \frac{lb}{ft}$$

Wall Lines DD:

$$v_{dd} \cdot \frac{Ldd}{33 \cdot ft} = 34.78 \frac{lb}{ft}$$

$$E_{dd} \cdot \frac{Ldd}{33 \cdot ft} = 43.44 \frac{lb}{ft}$$

Wall Lines HH:

$$v_{hh} \cdot \frac{Lhh}{74 \cdot ft} = 63.27 \frac{lb}{ft}$$

$$E_{hh} \cdot \frac{Lhh}{74 \cdot ft} = 68.26 \frac{lb}{ft}$$

Wall Lines A:

$$v_a \cdot \frac{La}{64 \cdot ft} = 130.09 \frac{lb}{ft}$$

$$E_a \cdot \frac{La}{64 \cdot ft} = 141.40 \frac{lb}{ft}$$

Wall Lines E:

$$v_e \cdot \frac{Le}{33 \cdot ft} = 108.83 \frac{lb}{ft}$$

$$E_e \cdot \frac{Le}{33 \cdot ft} = 110.41 \frac{lb}{ft}$$

Wall Lines B:

$$v_b \cdot \frac{Lb}{74 \cdot ft} = 149.73 \frac{lb}{ft}$$

$$E_b \cdot \frac{Lb}{74 \cdot ft} = 136.64 \frac{lb}{ft}$$

Wall Lines F:

$$v_f \cdot \frac{Lf}{30 \cdot ft} = 146.63 \frac{lb}{ft}$$

$$E_f \cdot \frac{Lf}{30 \cdot ft} = 150.26 \frac{lb}{ft}$$

Wall Lines C:

$$v_c \cdot \frac{Lc}{24 \cdot ft} = 91.44 \frac{lb}{ft}$$

$$E_c \cdot \frac{Lc}{24 \cdot ft} = 91.14 \frac{lb}{ft}$$

Wall Lines G:

$$v_g \cdot \frac{Lg}{33 \cdot ft} = 107.35 \frac{lb}{ft}$$

$$E_g \cdot \frac{Lg}{33 \cdot ft} = 110.75 \frac{lb}{ft}$$

Wall Lines D:

$$v_d \cdot \frac{Ld}{33 \cdot ft} = 81.04 \frac{lb}{ft}$$

$$E_d \cdot \frac{Ld}{33 \cdot ft} = 80.07 \frac{lb}{ft}$$

Wall Lines H:

$$v_h \cdot \frac{Lh}{17 \cdot ft} = 24.77 \frac{lb}{ft}$$

$$E_h \cdot \frac{Lh}{17 \cdot ft} = 19.62 \frac{lb}{ft}$$

Maximum Load For 6x6 DF#1 Wood Post

$$\overline{DSJ} := \frac{psi}{144}$$

$$\overline{DLJ} := psf \cdot ft$$

$$lb := plf \cdot ft$$

$$\overline{LJ} := 10 \cdot ft$$

$$F'_c := 1000 \cdot psi \quad C_D := 1 \quad C_{Fb} := 1 \quad C_M := 1 \quad C := 1 \quad C_L := 1 \quad C_{Fc} := 1$$

$$E' := 1600000 \cdot psi$$

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

Axial Load Capacity:

Slenderness Ratio (SL)

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

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 1008.33 \text{ psi}$$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F''_c}} \right) \cdot K_f = 0.69$$

$$F'_c := C_p \cdot F''_c \quad F'_c = 694 \text{ psi} \quad P_{max} := F'_c \cdot A \quad P_{max} = 20989 \text{ lb} \quad (\text{Maximum post Capacity})$$

6x6 Wood Post Properties

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

$$d := 5.5 \cdot in$$

$$t := 5.5 \cdot in$$

$$A := t \cdot h \quad A = 30.3 \text{ in}^2$$

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

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

Maximum Load For 6x6 HF#2 Treated Post

$$\overline{DSJ} := \frac{psi}{144}$$

$$\overline{DLJ} := psf \cdot ft$$

$$lb := plf \cdot ft$$

$$\overline{LJ} := 10 \cdot ft$$

$$F'_c := 460 \cdot psi \quad C_D := 1 \quad C_{Fb} := 1 \quad C_M := 1 \quad C := 1 \quad C_L := 1 \quad C_{Fc} := 1$$

$$E' := 1045000 \cdot psi$$

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

Axial Load Capacity:

Slenderness Ratio (SL)

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

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 658.57 \text{ psi}$$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F''_c}} \right) \cdot K_f = 0.80$$

$$F'_c := C_p \cdot F''_c \quad F'_c = 367 \text{ psi} \quad P_{max} := F'_c \cdot A \quad P_{max} = 11112 \text{ lb} \quad (\text{Maximum post Capacity})$$

6x6 Wood Post Properties

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

$$d := 5.5 \cdot in$$

$$t := 5.5 \cdot in$$

$$A := t \cdot h \quad A = 30.3 \text{ in}^2$$

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

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

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

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

$F_c := 800 \cdot psi$ $C_D := 1$ $C_{FB} := 1$ $C_M := 1$ $C := 1$ $C_L := 1$ $C_{Fc} := 1.1$

$E := 1200000 \cdot psi$

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

(3)2x6 Wood Post Properties

$K := 1$ ($K_t = 0.6$ for unbraced nailed built up posts - 0.75 for bolted)

$d := 5.5 \cdot in$

$l := (3) \cdot 1.5 \cdot in$

$A := t \cdot h$ $A = 24.8 \cdot in^2$

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

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

Axial Load Capacity:
 Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 756.25 \cdot psi$

$C_B := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{C}} \right) \cdot K_j = 0.64$

$F'_c := C_p \cdot F''_c$ $F'_c = 560 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 13863 \cdot lb$ (Maximum post Capacity)

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

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

$F_c := 800 \cdot psi$ $C_D := 1$ $C_{FB} := 1$ $C_M := 1$ $C := 1$ $C_L := 1$ $C_{Fc} := 1.1$

$E := 1200000 \cdot psi$

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

(2)2x6 Wood Post Properties

$K := 1$ ($K_t = 0.6$ for unbraced nailed built up posts - 0.75 for bolted)

$d := 5.5 \cdot in$

$l := (2) \cdot 1.5 \cdot in$

$A := t \cdot h$ $A = 16.5 \cdot in^2$

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

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

Axial Load Capacity:
 Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 756.25 \cdot psi$

$C_B := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{C}} \right) \cdot K_j = 0.64$

$F'_c := C_p \cdot F''_c$ $F'_c = 560 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 9242 \cdot lb$ (Maximum post Capacity)

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

$$psj := \frac{psi}{144}$$

$$dlj := psf \cdot ft$$

$$lbj := plf \cdot ft$$

$$Hj := 10 \cdot ft$$

$$F_c := 800 \cdot psi$$

$$C_D := 1$$

$$C_{FB} := 1$$

$$C_M := 1$$

$$C := 1$$

$$C_L := 1$$

$$C_{Fc} := 1.1$$

$$E := 1200000 \cdot psi$$

$$F''_c := F_c \cdot C_D \cdot C_{Fc}$$

$$F''_c = 880 \text{ psi}$$

Axial Load Capacity:

Slenderness Ratio (SL)

$$SL := \frac{H}{h}$$

$$C := 0.8$$

$$K_{CB} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 306.25 \text{ psi}$$

$$C_B := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{C}} \right) \cdot K_f = 0.32$$

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

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

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

$$P_{max} = 4411 \text{ lb}$$

(Maximum post Capacity)

(3)2x4 Wood Post Properties

$$K := 1$$

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

$$h := 3.5 \cdot in$$

$$l := (3) \cdot 1.5 \cdot in$$

$$A := t \cdot h$$

$$A = 15.8 \text{ in}^2$$

$$I := \frac{t \cdot h^3}{12}$$

$$I = 16.1 \text{ in}^4$$

$$S := \frac{I \cdot 2}{h}$$

$$S = 9.2 \text{ in}^3$$

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

$$psj := \frac{psi}{144}$$

$$dlj := psf \cdot ft$$

$$lbj := plf \cdot ft$$

$$Hj := 10 \cdot ft$$

$$F_c := 800 \cdot psi$$

$$C_D := 1$$

$$C_{FB} := 1$$

$$C_M := 1$$

$$C := 1$$

$$C_L := 1$$

$$C_{Fc} := 1.1$$

$$E := 1200000 \cdot psi$$

$$F''_c := F_c \cdot C_D \cdot C_{Fc}$$

$$F''_c = 880 \text{ psi}$$

Axial Load Capacity:

Slenderness Ratio (SL)

$$SL := \frac{H}{h}$$

$$C := 0.8$$

$$K_{CB} := 0.3$$

$$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 306.25 \text{ psi}$$

$$C_B := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{C}} \right) \cdot K_f = 0.32$$

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

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

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

$$P_{max} = 2941 \text{ lb}$$

(Maximum post Capacity)

(2)2x4 Wood Post Properties

$$K := 1$$

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

$$h := 3.5 \cdot in$$

$$l := (2) \cdot 1.5 \cdot in$$

$$A := t \cdot h$$

$$A = 10.5 \text{ in}^2$$

$$I := \frac{t \cdot h^3}{12}$$

$$I = 10.7 \text{ in}^4$$

$$S := \frac{I \cdot 2}{h}$$

$$S = 6.1 \text{ in}^3$$

Maximum Load For 4x4 HF#2 Treated Post

$$\overline{psf} := \frac{psi}{144}$$

$$\overline{plf} := psf \cdot ft$$

$$\overline{lb} := plf \cdot ft$$

$$\overline{H} := 6.25 \cdot ft$$

$$\overline{F'_c} := 1040 \cdot psi$$

$$\overline{C_D} := 1$$

$$\overline{C_{Fb}} := 1$$

$$\overline{C_M} := 1$$

$$\overline{C_1} := 1$$

$$\overline{C_2} := 1$$

$$\overline{C_{Fc}} := 1$$

$$\overline{E} := 1235000 \cdot psi$$

$$\overline{F''_c} := F'_c \cdot C_D \cdot C_{Fc}$$

$$F''_c = 1040 \cdot psi$$

Axial Load Capacity:

Slenderness Ratio (SL)

$$\overline{SL} := \frac{H}{h}$$

$$\overline{C} := 0.8$$

$$\overline{K_{CE}} := 0.3$$

$$\overline{F_{CE}} := \frac{K_{CE} \cdot E'}{SL^2} = 806.87 \cdot psi$$

$$\overline{C_B} := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{F_{CE}}{F''_c}} \right) \cdot K_f = 0.60$$

$$\overline{F'_d} := C_p \cdot F''_c$$

$$F'_c = 622 \cdot psi$$

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

$$P_{max} = 7618 \cdot lb$$

(Maximum post Capacity)

6x6 Wood Post Properties

$$\overline{K_1} := 1$$

($K_1 = 0.6$ for unbraced nailed built up posts - 0.75 for bolted)

$$\overline{d_1} := 3.5 \cdot in$$

$$\overline{d_2} := 3.5 \cdot in$$

$$\overline{A} := t \cdot h$$

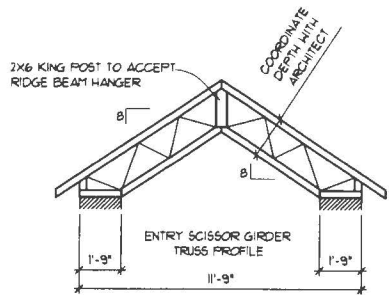
$$A = 12.3 \cdot in^2$$

$$\overline{I} := \frac{t \cdot h^3}{12}$$

$$I = 12.5 \cdot in^4$$

$$\overline{S} := \frac{I \cdot 2}{h}$$

$$S = 7.1 \cdot in^3$$



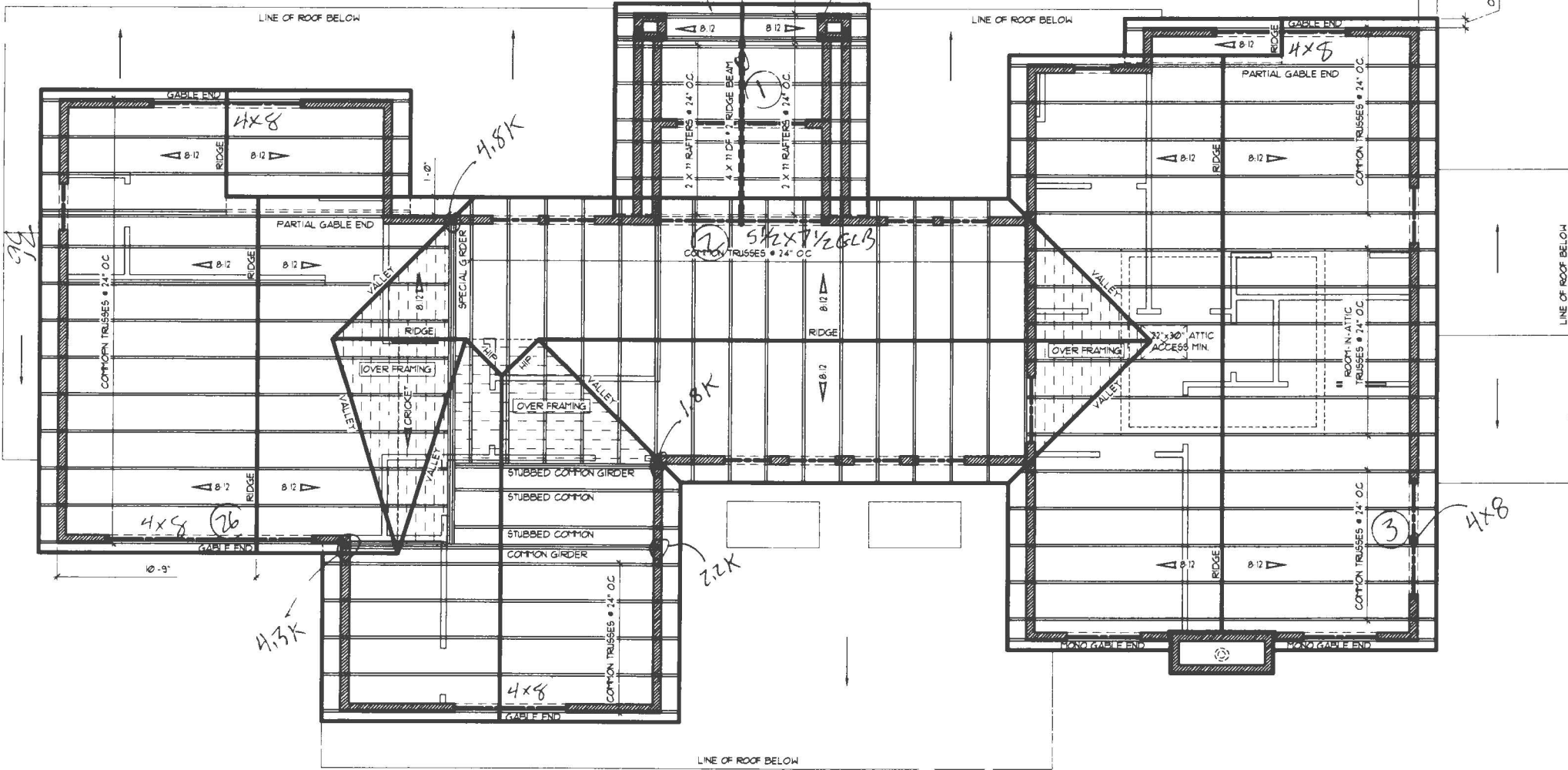
PARALLEL CHORD SCISSOR GABLE GIRDERS
 6x8 DF #1 OR (2) 2x10 UF #2

STONE VENEER AT GABLE

KG10 AT EA GIRDER

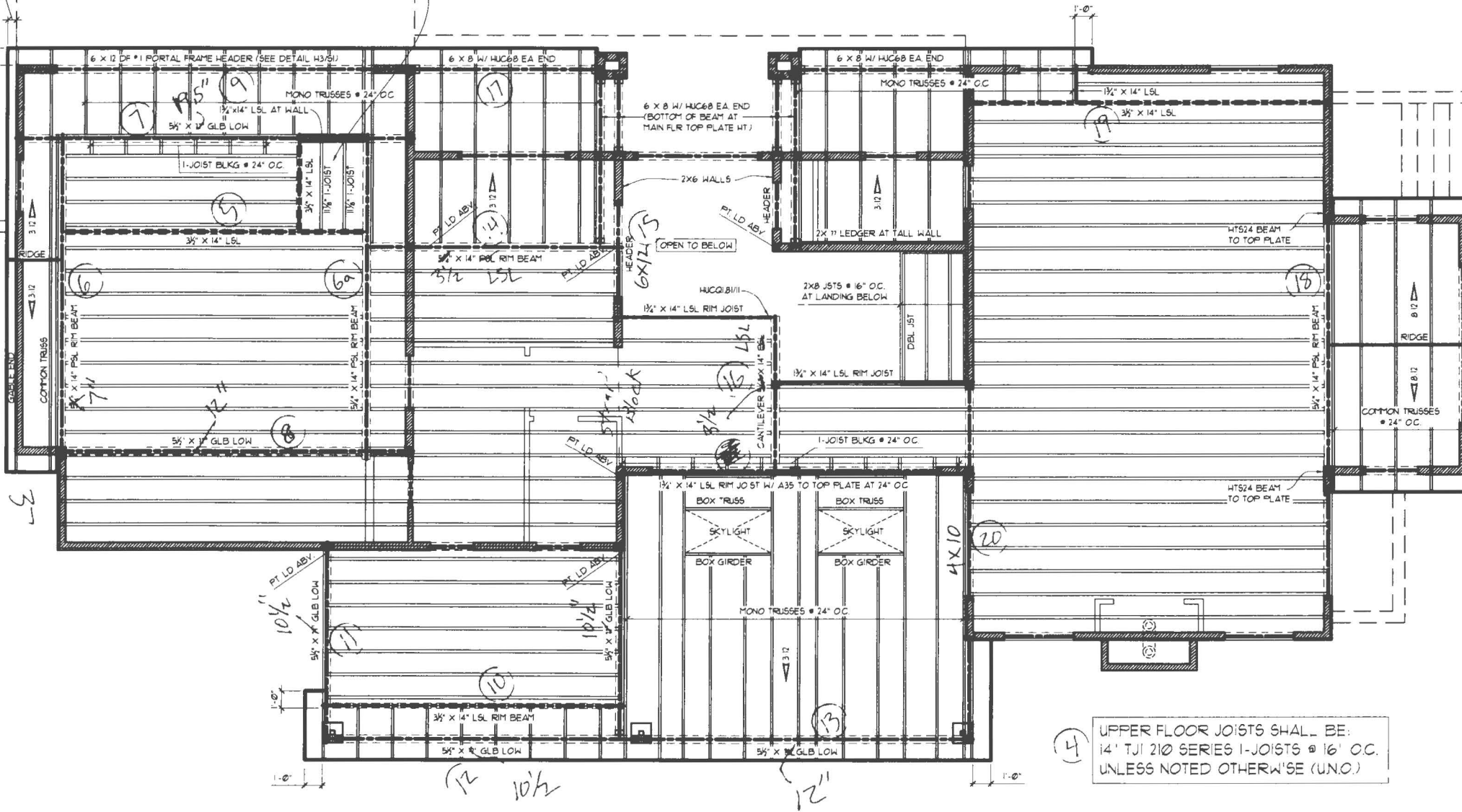
1'-0" OH TYP UNO

6" GABLE CH TYP UNO



LINE OF ROOF BELOW

DROPPED FRAMING FOR FLUSH ENTRY SHOWERS:
 PROVIDE 2X6 LEDGERS & BLOCKING AROUND
 PERIMETER TO ACCEPT EDGE NAILING. SECURE
 2X6 TO PERIMETER FRAMING W/ 10d COMMON
 NAILS (Ø148"x3") STAGGERED AT 6" OC.



4 UPPER FLOOR JOISTS SHALL BE:
 14' TJI 210 SERIES 1-JOISTS @ 16" OC.
 UNLESS NOTED OTHERWISE (U.N.O.)

LINE OF THICKENED SLAB EDGE
CONTINUOUS FOOTING

SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"x6" #6 WELDED WIRE MESH ON
10 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)
SLOPE 1/8" PER FT TOWARD DOOR

30
SLAB ON GRADE
4" CONCRETE SLAB OVER
4" COMPACTED GRANULAR FILL
SLOPE AWAY FROM BUILDING
30

SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"x6" #6 WELDED WIRE MESH ON
R-10 RIGID FOAM UNDER-SLAB RATED
INSULATION (F_c+20 PSI) MINIMUM OVER
10 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)

SLAB ON GRADE
4" CONCRETE SLAB OVER
4" COMPACTED GRANULAR FILL
SLOPE AWAY FROM BUILDING

POURED IN PLACE
CONCRETE STEPS
PROVIDE DRAINAGE
AT BOTTOM

8"x24" GRAWL
ACCESS MINIMUM

GRAWL SPACE
30" MINIMUM CLEARANCE (MIN.) AT BOTTOM
W/ 10 MIL VAPOR BARRIER MINIMUM
W/ SEALS OVERLAPPED 6" MINIMUM

WINDOW WELL
PROVIDE DRAINAGE

STEP TOP
OF WALL

STEP TOP
OF WALL

39

30

30

30

30

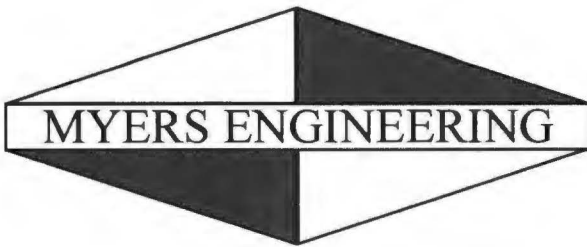
30

1'-0"

1'-0"

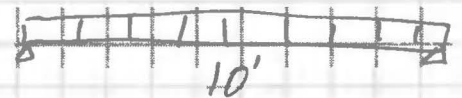
1'-0"

1'-4"

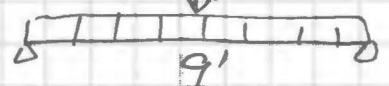


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① $w_D = 15 \text{ psf} \left(\frac{14'}{2}\right) = 105 \text{ plf}$
 $w_S = 25 \text{ psf} \left(\frac{14'}{2}\right) = 175 \text{ plf}$



6x8 DF #1
 OR (2) 2x10 DF #2



5 1/2 x 7 1/2 GLB

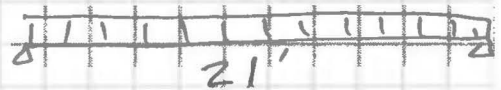
② $w_D = 15 \text{ psf} \left(\frac{16'}{2}\right) = 120 \text{ plf}$
 $w_S = 25 \text{ psf} \left(\frac{16'}{2}\right) = 200 \text{ plf}$



4x8 DF #2

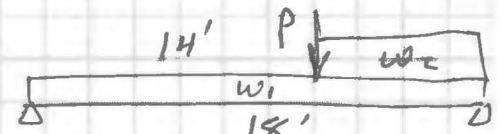
$P = 530 \# \text{DL} + 880 \# \text{SL}$ from ①

③ $w_D = 15 \text{ psf} \left(\frac{23'}{2}\right) = 172.5 \text{ plf}$
 $w_S = 25 \text{ psf} \left(\frac{23'}{2}\right) = 287.5 \text{ plf}$



14" TJI 210 series @ 16" o.c.

④ $w_D = 15 \text{ psf}$
 $w_L = 40 \text{ psf}$



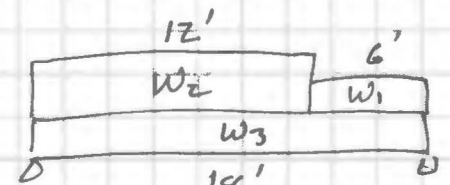
1 3/4 x 14 LVL

⑤ $w_{D1} = 15 \text{ plf}$
 $w_{L1} = 40 \text{ plf}$

$P = 300 \# \text{DL} + 770 \# \text{LL}$

$w_{D2} = 45 \text{ plf}$
 $w_{L2} = 120 \text{ plf}$

⑥ $w_{D1} = 15 \text{ psf} \left(\frac{20'}{2}\right) + 12 \text{ psf} (9') = 254 \text{ plf}$
 $w_{S1} = 25 \text{ psf} \left(\frac{20'}{2}\right) = 250 \text{ plf}$



7x14 PSL

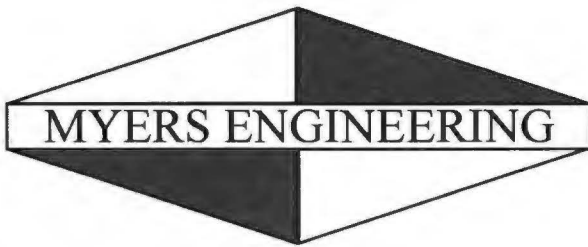
$w_{D2} = 15 \text{ psf} \left(\frac{23'}{2}\right) + 12 \text{ psf} (9') = 280.5 \text{ plf}$
 $w_{S2} = 25 \text{ psf} \left(\frac{23'}{2}\right) = 287.5 \text{ plf}$

$w_{D3} = 15 \text{ psf} \left(\frac{18'}{2} + 1'\right) = 150 \text{ plf}$
 $w_{L3} = 40 \text{ psf} \left(\frac{18'}{2}\right) = 360 \text{ plf}$

FOR RKK
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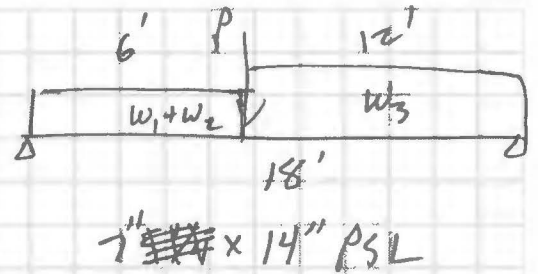
40

DATE 2-24-25
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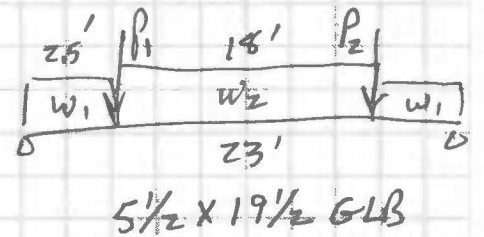
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6a) $w_{D1} = 25 \text{ psf}$
 $w_{S1} = 250 \text{ psf}$
 $w_{D2} = 15 \text{ psf} + 10 \text{ psf} = 123 \text{ psf}$
 $w_{L2} = 40 \text{ psf}$
 $P = 530 \text{ DL} + 1390 \text{ LL}$ from (5)



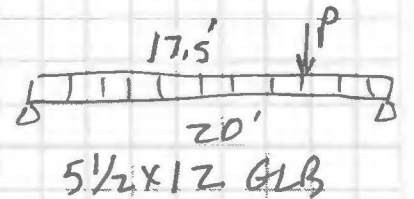
$w_{D3} = 15 \text{ psf} \left(\frac{20'}{2}\right) = 150 \text{ psf}$
 $w_{L3} = 40 \text{ psf} \left(\frac{20'}{2}\right) = 400 \text{ psf}$

7) $w_{D1} = 15 \text{ psf} \left(\frac{22'}{2}\right) (1.25) = 206.3 \text{ psf}$
 $w_{S1} = 25 \text{ psf} \left(\frac{22'}{2}\right) (1.25) = 343.8 \text{ psf}$
 $w_{D2} = 15 \text{ psf} \left(2' + \frac{1}{2}' + 1'\right) + 12 \text{ psf} (9') = 183 \text{ psf}$
 $w_{L2} = 40 \text{ psf}$
 $w_{S2} = 25 \text{ psf} \left(2' + \frac{1}{2}'\right) = 100 \text{ psf}$

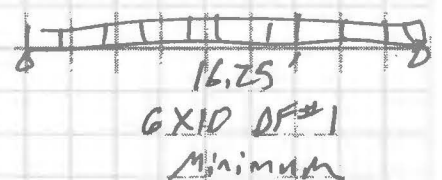


$P_1 = 3920 \text{ DL} + 3410 \text{ LL} + 2550 \text{ SL}$ from (6)
 $P_2 = 2860 \text{ DL} + 2730 \text{ LL} + 2400 \text{ SL}$ from (6a)

8) $w_D = 15 \text{ psf}$
 $w_L = 40 \text{ psf}$
 $P = 1760 \text{ DL} + 3700 \text{ LL} + 2550 \text{ SL}$ from (6a)



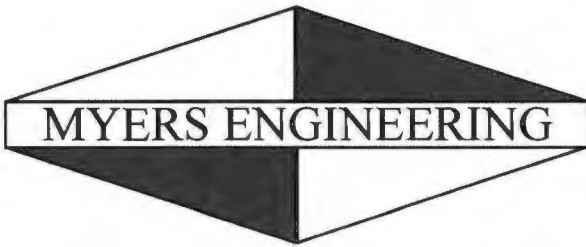
9) $w_D = 15 \text{ psf} \left(\frac{6'}{2}\right) = 45 \text{ psf}$
 $w_S = 25 \text{ psf} \left(\frac{6'}{2}\right) = 75 \text{ psf}$



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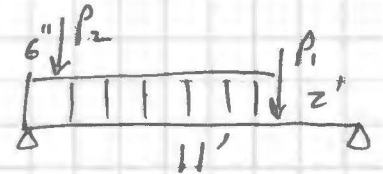


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(10) $w_D = 15 \text{ psf} (2' + 1' + 1') + 12 \text{ psf} (9') = 1646 \text{ plf}$
 $w_L = 40 \text{ plf}$
 $w_S = 25 \text{ psf} (2' + 1') = 75 \text{ plf}$



(11) $w_D = 15 \text{ psf} (17.5'/2 + 19.5'/2) + 12 \text{ psf} (9') = 386 \text{ plf}$
 $w_L = 40 \text{ psf} (17.5'/2) = 350 \text{ plf}$
 $w_S = 25 \text{ psf} (19.5'/2) = 244 \text{ plf}$



$P_1 = 1430 \text{# DL} + 640 \text{# SL}$ from (10)

5 1/2 x 10 1/2 GLB

$P_2 = 1740 \text{# DL} + 2560 \text{# SL}$ from girder truss

(12) $w_D = 15 \text{ psf} (4'/2) = 30 \text{ plf}$
 $w_S = 25 \text{ psf} (4'/2) = 50 \text{ plf}$



$P = 2670 \text{# DL} + 1290 \text{# LL} + 1540 \text{# SL}$ from (11)

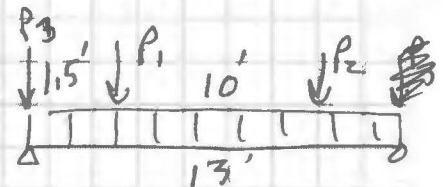
5 1/2 x 10 1/2 GLB

(13) $w_D = 15 \text{ psf} (17.5'/2) = 131.3 \text{ plf}$
 $w_S = 25 \text{ psf} (17.5'/2) = 218.8 \text{ plf}$



5 1/2 x 12 GLB

(14) $w_D = 15 \text{ psf} (15'/2 + 5'/2 + 1') + 12 \text{ psf} (9') = 273 \text{ plf}$
 $w_L = 40 \text{ plf}$
 $w_S = 25 \text{ psf} (15'/2 + 5'/2) = 250 \text{ plf}$



$P_1 = 530 \text{# DL} + 880 \text{# SL}$ from (1)

3 1/2 x 14 LSL

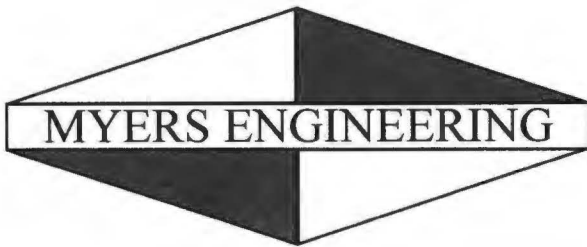
$P_2 = 1945 \text{# DL} + 2860 \text{# SL}$ from girder truss

$P_3 = 810 \text{# DL} + 1340 \text{# SL}$ from (2)

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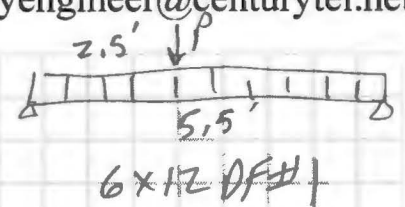
42

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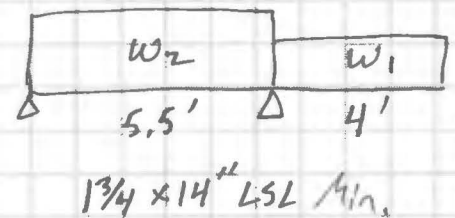


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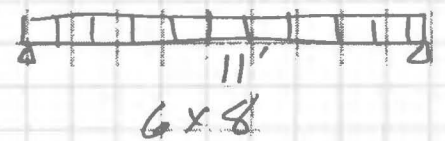
(15) $w_D = 15 \text{ psf} \left(\frac{13'}{2}\right) = 97.5 \text{ pIF}$
 $w_L = 40 \text{ psf} \left(\frac{13'}{2}\right) = 260 \text{ pIF}$
 $P_1 = 3260 \# \text{DL} + \cancel{1570 \# \text{AE}} + 4050 \# \text{SL from (14)}$



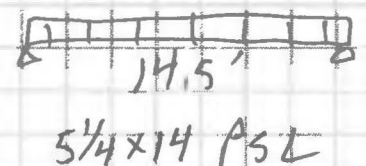
(16) $w_{D1} = 15 \text{ psf} \left(\frac{27'}{2}\right) = 202.5 \text{ pIF}$
 $w_{L1} = 40 \text{ psf} \left(\frac{27'}{2}\right) = 540 \text{ pIF}$
 $w_{D2} = 15 \text{ psf} \left(\frac{33'}{2}\right) = 247.5 \text{ pIF}$
 $w_{L2} = 40 \text{ psf} \left(\frac{33'}{2}\right) = 660 \text{ pIF}$



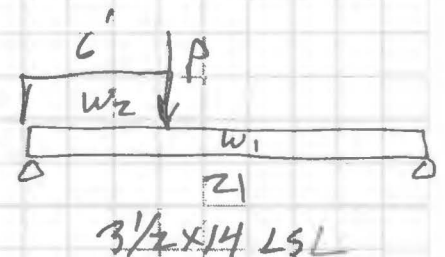
(17) $w_D = 15 \text{ psf} \left(\frac{7'}{2}\right) = 52.5 \text{ pIF}$
 $w_S = 25 \text{ psf} \left(\frac{7'}{2}\right) = 87.5 \text{ pIF}$



(18) $w_D = 15 \text{ psf} \left(\frac{23'}{2} + \frac{21'}{2} + 1'\right) + 12 \text{ psf} (9') = 453 \text{ pIF}$
 $w_L = 40 \text{ psf} \left(\frac{21'}{2}\right) = 420 \text{ pIF}$
 $w_S = 25 \text{ psf} \left(\frac{23'}{2}\right) = 287.5 \text{ pIF}$



(19) $w_{D1} = 15 \text{ pIF}$
 $w_{L1} = 40 \text{ pIF}$
 $w_{D2} = 15 \text{ psf} \left(\frac{2'}{2} + 2'\right) + 12 \text{ psf} (9') = 153 \text{ pIF}$
 $w_{S2} = 25 \text{ psf} \left(\frac{2'}{2} + 2'\right) = 75 \text{ pIF}$

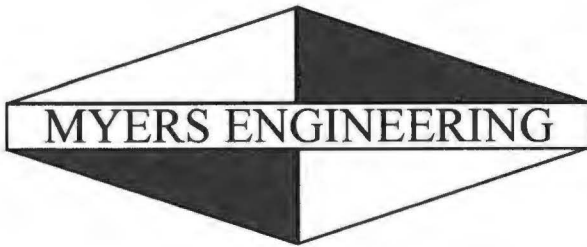


$P = 350 \# \text{DL} + 300 \# \text{DL} + 213 \# \text{SL from LSL}$

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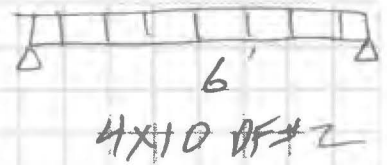
43

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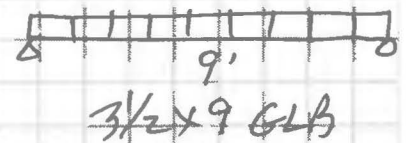


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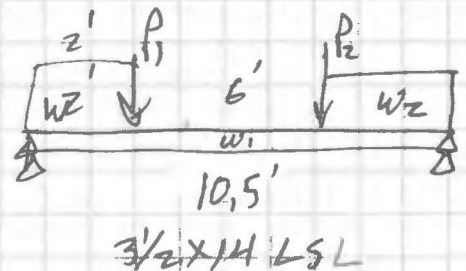
(20) $w_D = 15 \text{ psf} \left(23\frac{1}{2} + 21\frac{1}{2} \right) + 12 \text{ psf} (9') = 438 \text{ plf}$
 $w_L = 40 \text{ psf} \left(21\frac{1}{2} \right) = 420 \text{ plf}$
 $w_S = 25 \text{ psf} \left(21\frac{1}{2} \right) = 288 \text{ plf}$



(21) $w_D = 15 \text{ psf} \left(22\frac{1}{2} \right) = 165 \text{ plf}$
 $w_L = 40 \text{ psf} \left(22\frac{1}{2} \right) = 440 \text{ plf}$



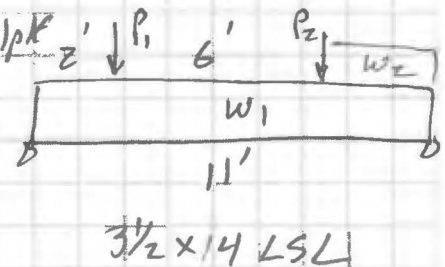
(22) $w_{D1} = 15 \text{ plf}$
 $w_{L1} = 40 \text{ plf}$
 $w_{D2} = 15 \text{ psf} \left(21\frac{1}{2} \right) = 157.5 \text{ plf}$
 $w_{L2} = 40 \text{ psf} \left(21\frac{1}{2} \right) = 420 \text{ plf}$



$P_1 = 2190^{\#} \text{ DL} + 780^{\#} \text{ LL} + 2360^{\#} \text{ SL}$ from (15)

$P_2 = 1650^{\#} \text{ DL} + 780^{\#} \text{ LL} + 1690^{\#} \text{ SL}$ from (15)

(23) $w_{D1} = 15 \text{ psf} \left(1' + 5\frac{1}{2} + 16\frac{1}{2} \right) + 10 \text{ psf} (10') + 12 \text{ psf} (9') = 381 \text{ plf}$
 $w_{L1} = 40 \text{ plf}$
 $w_{S1} = 25 \text{ psf} \left(16\frac{1}{2} + 5\frac{1}{2} \right) = 263 \text{ plf}$



$w_{D2} = 15 \text{ psf} \left(8\frac{1}{2} \right) = 60 \text{ plf}$
 $w_{L2} = 40 \text{ psf} \left(6\frac{1}{2} \right) = 160 \text{ plf}$

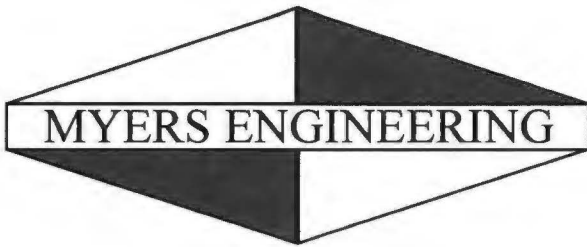
$P_1 = 530^{\#} \text{ DL} + 880^{\#} \text{ SL}$ from (1)

$P_2 = 105^{\#} \text{ DL} + 280^{\#} \text{ LL}$ from stair

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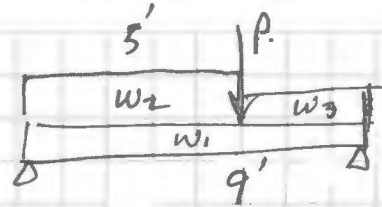
(24) $w_{D1} = 15 \text{ p/f}$
 $w_{L1} = 40 \text{ p/f}$

$w_{D2} = 15 \text{ p/f} (11\frac{1}{2}) = 83 \text{ p/f}$
 $w_{L2} = 40 \text{ p/f} (11\frac{1}{2}) = 220 \text{ p/f}$

$w_{D3} = 15 \text{ p/f} (7\frac{1}{2}) = 53 \text{ p/f}$
 $w_{L3} = 40 \text{ p/f} (7\frac{1}{2}) = 140 \text{ p/f}$

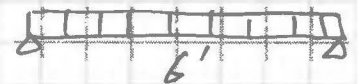
$P_1 = 2580 \text{ DL} + 360 \text{ LL} + 2170 \text{ SL}$ from (23)

$P_2 = 810 \text{ DL} + 1340 \text{ SL}$ from (2)



3/2 x 14 LSL

(25) $w_D = 15 \text{ p/f} (23\frac{1}{2} + 21\frac{1}{2} + 21\frac{1}{2}) + 12 \text{ p/f} (9' + 10') = 716 \text{ p/f}$
 $w_L = 40 \text{ p/f} (21\frac{1}{2} + 21\frac{1}{2}) = 840 \text{ p/f}$
 $w_S = 25 \text{ p/f} (23\frac{1}{2}) = 2846 \text{ p/f}$



3/2 x 9 GLB

(26) $w_D = 15 \text{ p/f} (2') + 12 \text{ p/f} (5') = 90 \text{ p/f}$
 $w_S = 25 \text{ p/f} (2') = 50 \text{ p/f}$



4 x 8

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Multiple Simple Beam

Project File: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build 20.25.02.04

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Description :

Wood Beam Design : 1. Entry Ridge beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 6x8, 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	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

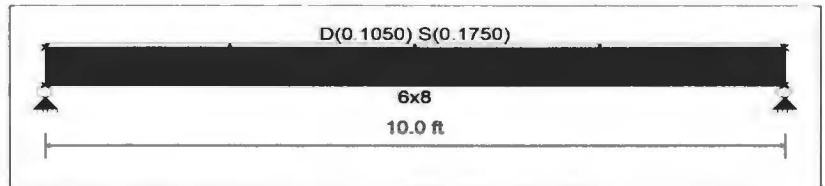
Unif Load: D = 0.1050, S = 0.1750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.525 : 1**
 fb : Actual : 814.55 psi at 5.000 ft in Span # 1
 Fb : Allowable : 1,552.50 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.229 : 1**
 fv : Actual : 44.80 psi at 9.400 ft in Span # 1
 Fv : Allowable : 195.50 psi
 Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.53			0.88			
Right Support	0.53			0.88			



Max Deflections

Transient Downward	0.128 in	Total Downward	0.205 in
Ratio	937	Ratio	586
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 : 1. Entry Ridge beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 2-2x10, 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

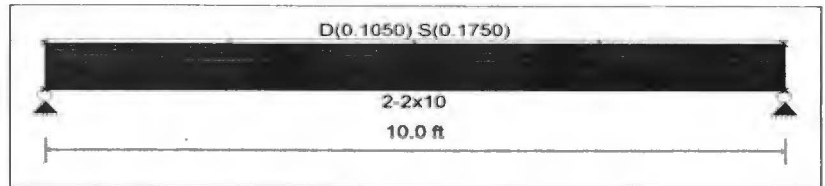
Unif Load: D = 0.1050, S = 0.1750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.862 : 1**
 fb : Actual : 981.74 psi at 5.000 ft in Span # 1
 Fb : Allowable : 1,138.50 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.310 : 1**
 fv : Actual : 64.07 psi at 9.233 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	0.53			0.88			
Right Support	0.53			0.88			



Max Deflections

Transient Downward	0.125 in	Total Downward	0.200 in
Ratio	959	Ratio	599
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: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build:20.25.02.04

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Wood Beam Design : 2. Header over Foyer

Calculations per NDS 2018, IBC 2018, CBC 2019

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	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

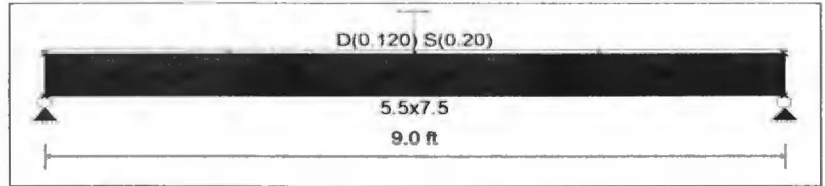
Unif Load: D = 0.120, S = 0.20 k/ft, Trib= 1.0 ft
 1Point: D = 0.530, S = 0.880 k @ 4.50 ft

Design Summary

Max fb/Fb Ratio = **0.543 : 1**
 fb : Actual : 1,492.36 psi at 4.500 ft in Span # 1
 Fb : Allowable : 2,746.17 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.233 : 1**
 fv : Actual : 71.02 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	0.81			1.34			
Right Support	0.81			1.34			



Max Deflections

Transient Downward	0.152 in	Total Downward	0.243 in
Ratio	710	Ratio	443
	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 Bed-2

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 4x8, 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

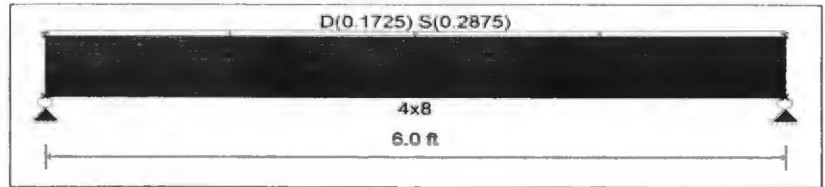
Unif Load: D = 0.1725, S = 0.2875 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.608 : 1**
 fb : Actual : 810.14 psi at 3.000 ft in Span # 1
 Fb : Allowable : 1,333.02 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.315 : 1**
 fv : Actual : 65.26 psi at 5.400 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	0.52			0.86			
Right Support	0.52			0.86			



Max Deflections

Transient Downward	0.047 in	Total Downward	0.076 in
Ratio	1519	Ratio	949
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

FLOOR SPAN TABLES

9 1/2" - 18" JOISTS

4

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9 1/2"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11 1/2"	110	20'-2"	18'-5"	17'-4"	15'-9" ⁽¹⁾	20'-2"	17'-8"	16'-1" ⁽¹⁾	14'-4" ⁽¹⁾
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9" ⁽¹⁾
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7" ⁽¹⁾
14"	110	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10" ⁽¹⁾
	210	22'-10"	20'-11"	19'-2"	17'-2" ⁽¹⁾	22'-2"	19'-2"	17'-6" ⁽¹⁾	15'-0" ⁽¹⁾
	230	23'-11"	21'-10"	20'-8"	18'-10" ⁽¹⁾	23'-11"	21'-1"	19'-2" ⁽¹⁾	16'-7" ⁽¹⁾
18"	110	24'-8"	22'-6"	21'-2"	19'-9" ⁽¹⁾	24'-8"	22'-2"	20'-3" ⁽¹⁾	17'-6" ⁽¹⁾
	210	25'-4"	23'-8"	22'-4"	20'-9" ⁽¹⁾	26'-0"	23'-8"	22'-4" ⁽¹⁾	17'-10" ⁽¹⁾
	230	29'-6"	26'-10"	25'-4"	23'-6"	29'-6"	26'-10"	25'-4" ⁽¹⁾	20'-11" ⁽¹⁾

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

- Tables are based on:
 - Uniform loads.
 - More restrictive of simple or continuous span.
 - Clear distance between supports.
 - Minimum bearing length of 1 3/4" end (no web stiffeners) and 3 1/2" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9 1/2"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	20'-3"	18'-1"	16'-6"	14'-9"
11 1/2"	110	22'-3"	19'-4"	17'-8"	15'-9" ⁽¹⁾	20'-5"	17'-8"	16'-1" ⁽¹⁾	14'-4" ⁽¹⁾
	210	23'-4"	21'-2"	19'-4"	17'-3" ⁽¹⁾	22'-4"	19'-4"	17'-8"	15'-9" ⁽¹⁾
	230	24'-0"	21'-11"	20'-5"	18'-3"	23'-7"	20'-5"	18'-7"	16'-7" ⁽¹⁾
14"	110	25'-4"	23'-2"	21'-10"	20'-4" ⁽¹⁾	25'-4"	23'-2"	21'-10" ⁽¹⁾	17'-10" ⁽¹⁾
	210	24'-4"	21'-0"	19'-2"	17'-2" ⁽¹⁾	26'-10"	23'-3"	24'-9"	20'-11" ⁽¹⁾
	230	26'-6"	23'-1"	21'-1"	18'-10" ⁽¹⁾	24'-4"	21'-1"	19'-2" ⁽¹⁾	16'-7" ⁽¹⁾
18"	110	27'-3"	24'-4"	22'-2"	19'-10" ⁽¹⁾	25'-8"	22'-2"	20'-3" ⁽¹⁾	17'-6" ⁽¹⁾
	210	28'-9"	26'-3"	24'-9" ⁽¹⁾	21'-5" ⁽¹⁾	28'-9"	26'-3" ⁽¹⁾	22'-4" ⁽¹⁾	17'-10" ⁽¹⁾
	230	32'-8"	29'-9"	28'-0"	25'-2" ⁽¹⁾	32'-8"	29'-9"	26'-3" ⁽¹⁾	20'-11" ⁽¹⁾

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is less than 5/4" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		17'-9"	14'-2"	
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
380			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

• Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use TJI Pro™ Ratings included in FORTWELL™ and our span table web application.

These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

Multiple Simple Beam

Project File: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build: 20.25.02.04

MYERS ENGINEERING

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Wood Beam Design : 5. Floor beam at shower box out

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 1.75x14, TimberStrand LSL, Fully Braced

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

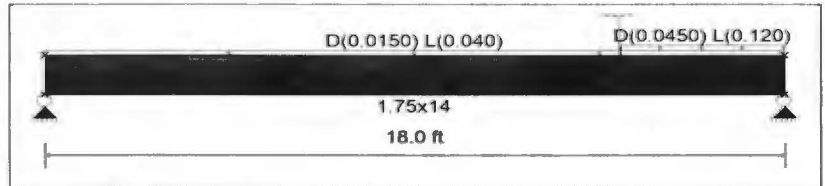
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
 Unif Load: D = 0.0450, L = 0.120 k/ft, 14.0 to 18.0 ft, Trib= 1.0 ft
 1Point: D = 0.30, L = 0.770 k @ 14.0 ft

Design Summary

Max fb/Fb Ratio = **0.483 : 1**
 fb : Actual : 1,237.40 psi at 13.980 ft in Span # 1
 Fb : Allowable : 2,563.39 psi
 Load Comb : +D+L

Max fv/Fv Ratio = **0.357 : 1**
 fv : Actual : 101.82 psi at 16.860 ft in Span # 1
 Fv : Allowable : 285.00 psi
 Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.22		0.58				
Right Support	0.53		1.39				



Max Deflections

Transient Downward	Ratio	Total Downward	Ratio
0.303 in	713	0.418 in	516
LC: L Only		LC: +D+L	
Transient Upward	Ratio	Total Upward	Ratio
0.000 in	9999	0.000 in	9999
LC:		LC:	

Wood Beam Design : 6. Side Wall support Beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 7x14, Parallam PSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : Parallam PSL 2.0E

Fb - Tension : 2,900.0 psi Fc - Prll : 2,900.0 psi Fv : 290.0 psi Ebend- xx : 2,000.0 ksi Density : 45.070 pcf
 Fb - Compr : 2,900.0 psi Fc - Perp : 750.0 psi Ft : 2,025.0 psi Eminbend - xx : 1,016.54 ksi

Applied Loads

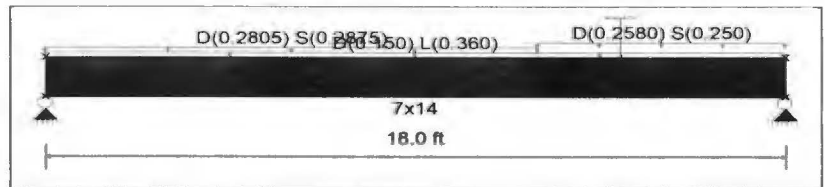
Unif Load: D = 0.150, L = 0.360 k/ft, Trib= 1.0 ft
 Unif Load: D = 0.2805, S = 0.2875 k/ft, 0.0 to 12.0 ft, Trib= 1.0 ft
 Unif Load: D = 0.2580, S = 0.250 k/ft, 12.0 to 18.0 ft, Trib= 1.0 ft
 1Point: D = 0.30, L = 0.770 k @ 14.0 ft

Design Summary

Max fb/Fb Ratio = **0.626 : 1**
 fb : Actual : 1,783.30 psi at 9.300 ft in Span # 1
 Fb : Allowable : 2,850.80 psi
 Load Comb : +D+L

Max fv/Fv Ratio = **0.367 : 1**
 fv : Actual : 106.51 psi at 16.860 ft in Span # 1
 Fv : Allowable : 290.00 psi
 Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.92		3.41	2.55			
Right Support	4.00		3.84	2.40			



Max Deflections

Transient Downward	Ratio	Total Downward	Ratio
0.299 in	723	0.706 in	305
LC: L Only		LC: +D+0.750L+0.750S	
Transient Upward	Ratio	Total Upward	Ratio
0.000 in	9999	0.000 in	9999
LC:		LC:	

Multiple Simple Beam

Project File: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build:20.25.02.04

MYERS ENGINEERING

(c) ENERCALC, LLC 1982-2025

Wood Beam Design : 6a. Side Wall support Beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 7x14, Parallam PSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : Parallam PSL 2.0E

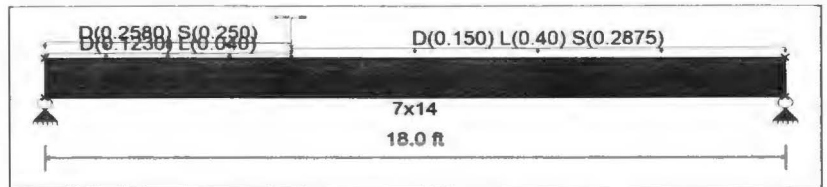
Fb - Tension 2,900.0 psi Fc - Prll 2,900.0 psi Fv 290.0 psi Ebend- xx 2,000.0 ksi Density 45.070 pcf
 Fb - Compr 2,900.0 psi Fc - Perp 750.0 psi Ft 2,025.0 psi Eminbend - xx 1,016.54 ksi

Applied Loads

Unif Load: D = 0.1230, L = 0.040 k/ft, 0.0 ft to 6.0 ft, Trib= 1.0 ft
 Unif Load: D = 0.150, L = 0.40, S = 0.2875 k/ft, 6.0 to 18.0 ft, Trib= 1.0 ft
 Unif Load: D = 0.2580, S = 0.250 k/ft, 0.0 to 6.0 ft, Trib= 1.0 ft
 1Point: D = 0.530, L = 1.390 k @ 6.0 ft

Design Summary

Max fb/Fb Ratio = **0.500 : 1**
 fb : Actual : 1,638.78 psi at 8.340 ft in Span # 1
 Fb : Allowable : 3,278.42 psi
 Load Comb : +D+0.750L+0.750S
 Max fv/FvRatio = **0.276 : 1**
 fv : Actual : 92.16 psi at 0.000 ft in Span # 1
 Fv : Allowable : 333.50 psi
 Load Comb : +D+0.750L+0.750S



Max Reactions (k) D Lr L S W E H
 Left Support 2.86 2.73 2.40
 Right Support 1.76 3.70 2.55

Max Deflections

Transient Downward 0.309 in Total Downward 0.570 in
 Ratio 699 Ratio 379
 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:

Wood Beam Design : 7. Beam over Front of Garage

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 5.5x19.5, GLB, Fully Braced

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

Wood Species : DF/DF

Wood Grade : 24F-V4

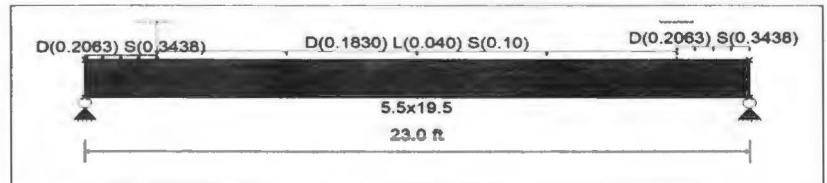
Fb - Tension 2,400.0 psi Fc - Prll 1,650.0 psi Fv 265.0 psi Ebend- xx 1,800.0 ksi Density 31.210 pcf
 Fb - Compr 1,850.0 psi Fc - Perp 650.0 psi Ft 1,100.0 psi Eminbend - xx 950.0 ksi

Applied Loads

Unif Load: D = 0.2063, S = 0.3438 k/ft, 0.0 ft to 2.50 ft, Trib= 1.0 ft
 Unif Load: D = 0.1830, L = 0.040, S = 0.10 k/ft, 2.50 to 20.50 ft, Trib= 1.0 ft
 Unif Load: D = 0.2063, S = 0.3438 k/ft, 20.50 to 23.0 ft, Trib= 1.0 ft
 1Point: D = 3.920, L = 3.410, S = 2.550 k @ 2.50 ft
 2Point: D = 2.860, L = 2.730, S = 2.40 k @ 20.50 ft

Design Summary

Max fb/Fb Ratio = **0.513 : 1**
 fb : Actual : 1,326.28 psi at 10.887 ft in Span # 1
 Fb : Allowable : 2,587.06 psi
 Load Comb : +D+0.750L+0.750S
 Max fv/FvRatio = **0.515 : 1**
 fv : Actual : 156.81 psi at 0.000 ft in Span # 1
 Fv : Allowable : 304.75 psi
 Load Comb : +D+0.750L+0.750S



Max Reactions (k) D Lr L S W E H
 Left Support 5.97 3.70 4.29
 Right Support 5.14 3.16 4.18

Max Deflections

Transient Downward 0.232 in Total Downward 0.657 in
 Ratio 1188 Ratio 419
 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:

Multiple Simple Beam

Project File: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build:20.25.02.04

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Wood Beam Design : 8. Beam over Rear of Garage

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 5.5x12, 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

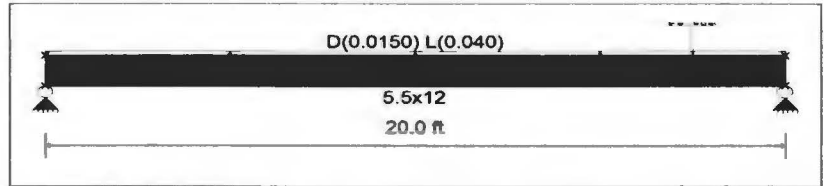
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
 1Point: D = 1.760, L = 3.70, S = 2.550 k @ 17.50 ft

Design Summary

Max fb/Fb Ratio = **0.499 : 1**
 fb : Actual : 1,194.35 psi at 17.467 ft in Span # 1
 Fb : Allowable : 2,394.77 psi
 Load Comb : +D+L

Max fv/FvRatio = **0.453 : 1**
 fv : Actual : 119.91 psi at 19.067 ft in Span # 1
 Fv : Allowable : 265.00 psi
 Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.37		0.86	0.32			
Right Support	1.69		3.64	2.23			



Max Deflections

Transient Downward	0.382 in	Total Downward	0.604 in
Ratio	628	Ratio	397
		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:

Wood Beam Design : 9. Garage Door Header

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 6x10, Sawn, Fully Unbraced

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

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

Fb - Tension 1350 psi Fc - Prll 925 psi Fv 170 psi Ebend- xx 1600 ksi Density 31.21 pcf
 Fb - Compr 1350 psi Fc - Perp 625 psi Ft 675 psi Eminbend - xx 580 ksi

Applied Loads

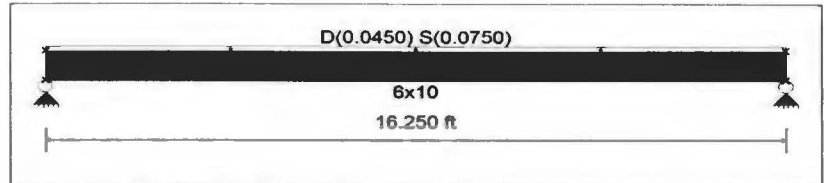
Unif Load: D = 0.0450, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.376 : 1**
 fb : Actual : 574.54 psi at 8.125 ft in Span # 1
 Fb : Allowable : 1,527.41 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.130 : 1**
 fv : Actual : 25.38 psi at 15.492 ft in Span # 1
 Fv : Allowable : 195.50 psi
 Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.37			0.61			
Right Support	0.37			0.61			



Max Deflections

Transient Downward	0.188 in	Total Downward	0.301 in
Ratio	1036	Ratio	647
		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: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build: 20.25.02.26

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Description :

Wood Beam Design : 10. Rear Rim beam at Bed-3

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	45.010 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

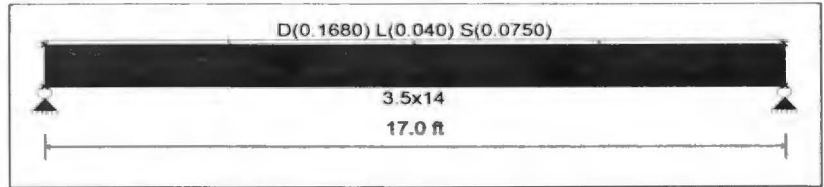
Unif Load: D = 0.1680, L = 0.040, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.366 : 1**
 fb : Actual : 964.00 psi at 8.500 ft in Span # 1
 Fb : Allowable : 2,636.10 psi
 Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.161 : 1**
 fv : Actual : 57.34 psi at 15.867 ft in Span # 1
 Fv : Allowable : 356.50 psi
 Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.43		0.34	0.64			
Right Support	1.43		0.34	0.64			



Max Deflections

Transient Downward	0.114 in	Total Downward	0.387 in
Ratio	1785	Ratio	526
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 : 11. Side beam at Bed-3

Calculations per NDS 2018, IBC 2018, CBC 2019

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 = 0.3860, L = 0.350, S = 0.2440 k/ft, 0.0 ft to 9.0 ft, Trib= 1.0 ft

1Point: D = 1.740, S = 2.560 k @ 0.50 ft

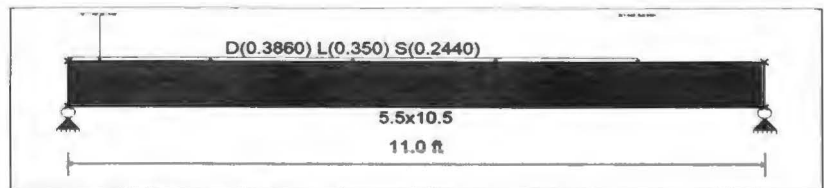
2Point: D = 1.430, S = 0.640 k @ 9.0 ft

Design Summary

Max fb/Fb Ratio = **0.627 : 1**
 fb : Actual : 1,730.07 psi at 5.537 ft in Span # 1
 Fb : Allowable : 2,760.00 psi
 Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.408 : 1**
 fv : Actual : 124.43 psi at 9.020 ft in Span # 1
 Fv : Allowable : 304.75 psi
 Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.97		1.86	3.86			
Right Support	2.67		1.29	1.54			



Max Deflections

Transient Downward	0.113 in	Total Downward	0.341 in
Ratio	1173	Ratio	386
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:	

Multiple Simple Beam

Project File: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build:20 25 02.26

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Wood Beam Design : 12. Beam at rear Patio behind Bed-3

Calculations per NDS 2018, IBC 2018, CBC 2019

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

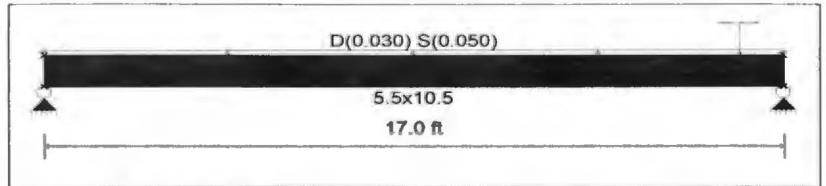
Unif Load: D = 0.030, S = 0.050 k/ft, Trib= 1.0 ft
 1Point: D = 2.670, L = 1.290, S = 1.540 k @ 16.0 ft

Design Summary

Max fb/Fb Ratio = 0.233 : 1
 fb : Actual : 643.96 psi at 12.693 ft in Span # 1
 Fb : Allowable : 2,760.00 psi
 Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = 0.428 : 1
 fv : Actual : 130.57 psi at 16.150 ft in Span # 1
 Fv : Allowable : 304.75 psi
 Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.41		0.08	0.52			
Right Support	2.77		1.21	1.87			



Max Deflections

Transient Downward	0.150 in	Total Downward	0.297 in
Ratio	1363	Ratio	686
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 : 13. Beam at rear Patio

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 5.5x12, 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

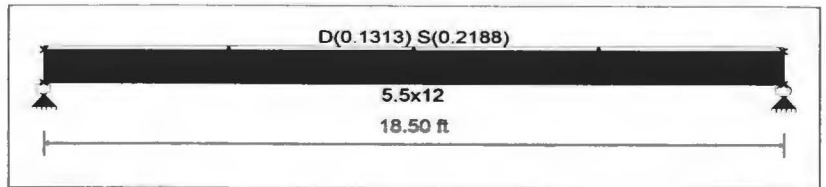
Unif Load: D = 0.1313, S = 0.2188 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.493 : 1
 fb : Actual : 1,361.61 psi at 9.250 ft in Span # 1
 Fb : Allowable : 2,760.00 psi
 Load Comb : +D+S

Max fv/FvRatio = 0.216 : 1
 fv : Actual : 65.75 psi at 17.513 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	1.21			2.02			
Right Support	1.21			2.02			



Max Deflections

Transient Downward	0.407 in	Total Downward	0.651 in
Ratio	545	Ratio	341
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: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build:20.25.02.26

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Wood Beam Design : 14. Floor beam supporting wall of Primary Closet

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

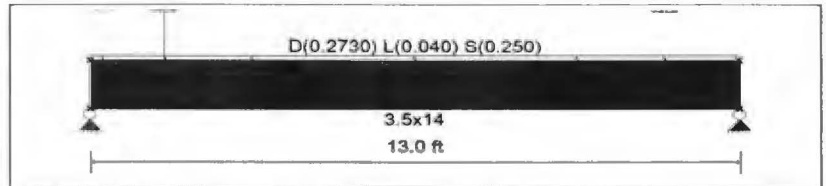
Fb - Tension	2325 psi	Fc - Prll	2050 psi	Fv	310 psi	Ebend- xx	1550 ksi	Density	45.01 pcf
Fb - Compr	2325 psi	Fc - Perp	800 psi	Ft	1070 psi	Eminbend - xx	787.815 ksi		

Applied Loads

Unif Load: D = 0.2730, L = 0.040, S = 0.250 k/ft, Trib= 1.0 ft
 1Point: D = 0.530, S = 0.880 k @ 1.50 ft
 2Point: D = 1.945, S = 2.860 k @ 11.50 ft
 3Point: D = 0.810, S = 1.340 k @ 0.250 ft

Design Summary

Max fb/Fb Ratio = **0.641 : 1**
 fb : Actual : 1,689.34 psi at 7.150 ft in Span # 1
 Fb : Allowable : 2,636.10 psi
 Load Comb : +D+S
 Max fv/FvRatio = **0.624 : 1**
 fv : Actual : 222.39 psi at 11.873 ft in Span # 1
 Fv : Allowable : 356.50 psi
 Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.26		0.26	4.05			
Right Support	3.57		0.26	4.28			

Max Deflections			
Transient Downward	0.217 in	Total Downward	0.416 in
Ratio	719	Ratio	374
	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 : 15. Header at Office Door

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 6x12, Sawn, Fully Unbraced

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

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.09750, L = 0.260 k/ft, Trib= 1.0 ft
 1Point: D = 3.260, S = 4.050 k @ 2.50 ft

Design Summary

Max fb/Fb Ratio = **0.712 : 1**
 fb : Actual : 1,097.46 psi at 2.500 ft in Span # 1
 Fb : Allowable : 1,541.50 psi
 Load Comb : +D+S
 Max fv/FvRatio = **0.542 : 1**
 fv : Actual : 105.89 psi at 0.000 ft in Span # 1
 Fv : Allowable : 195.50 psi
 Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.19		0.78	2.36			
Right Support	1.65		0.78	1.69			

Max Deflections			
Transient Downward	0.027 in	Total Downward	0.052 in
Ratio	2631	Ratio	1386
	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: 4115 78th AVE SE.ec6

LIC#: KW-06015659, Build: 20.25.02.26

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Wood Beam Design : 16. Cantilever beam at stair

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 1.75x14, TimberStrand LSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

Fb - Tension 2,325.0 psi Fc - Prll 2,050.0 psi Fv 310.0 psi Ebend- xx 1,550.0 ksi Density 45.010 pcf
 Fb - Compr 2,325.0 psi Fc - Perp 800.0 psi Ft 1,070.0 psi Eminbend - xx 787.82 ksi

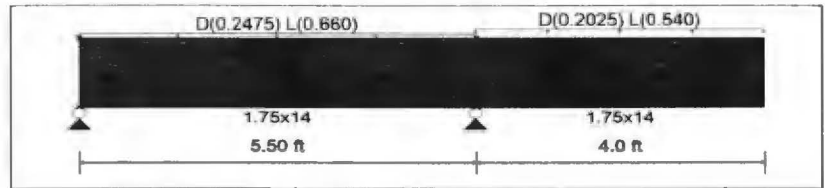
Applied Loads

Unif Load: D = 0.2475, L = 0.660 k/ft, 0.0 ft to 5.50 ft, Trib= 1.0 ft
 Unif Load: D = 0.2025, L = 0.540 k/ft, 5.50 to 9.50 ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.544 : 1**
 fb : Actual : 1,246.88 psi at 5.500 ft in Span # 1
 Fb : Allowable : 2,292.26 psi
 Load Comb : +D+L+H, LL Comb Run (*L)
 Max fv/FvRatio = **0.499 : 1**
 fv : Actual : 154.74 psi at 4.345 ft in Span # 1
 Fv : Allowable : 310.00 psi
 Load Comb : +D+L+H, LL Comb Run (LL)
 Max Reactions (k)

	D	Lr	L	S	W	E
Left Support	0.39		1.82			
Right Support	1.79		4.76			



Max Deflections

Transient Downward	0.136 in	Total Downward	0.168 in
Ratio	704	Ratio	570
L Only, LL Comb Run (*L)		+L+H, LL Comb Run (*L)	
Transient Upward	-0.051 in	Total Upward	-0.024 in
Ratio	1882	Ratio	2709
L Only, LL Comb Run (L*)		+L+H, LL Comb Run (*L)	

Wood Beam Design : 17. Porch Roof Beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 6x8, 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 875.0 psi Fc - Prll 600.0 psi Fv 170.0 psi Ebend- xx 1,300.0 ksi Density 31.210 pcf
 Fb - Compr 875.0 psi Fc - Perp 625.0 psi Ft 425.0 psi Eminbend - xx 470.0 ksi

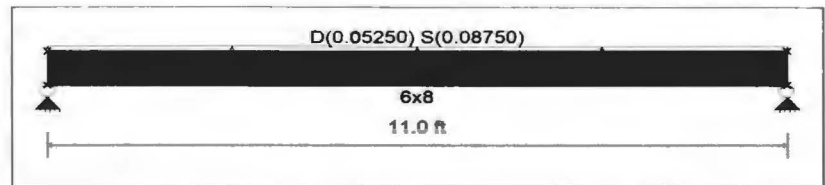
Applied Loads

Unif Load: D = 0.05250, S = 0.08750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.490 : 1**
 fb : Actual : 492.80 psi at 5.500 ft in Span # 1
 Fb : Allowable : 1,006.25 psi
 Load Comb : +D+S
 Max fv/FvRatio = **0.127 : 1**
 fv : Actual : 24.83 psi at 0.000 ft in Span # 1
 Fv : Allowable : 195.50 psi
 Load Comb : +D+S
 Max Reactions (k)

	D	Lr	L	S	W	E
Left Support	0.29			0.48		
Right Support	0.29			0.48		



Max Deflections

Transient Downward	0.115 in	Total Downward	0.184 in
Ratio	1145	Ratio	715
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 : 18. Rim beam Grid 7

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 5.25x14.0, Parallam PSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : Parallam PSL 2.0E

Fb - Tension 2,900.0 psi

Fc - Prll 2,900.0 psi

Fv 290.0 psi

Ebend- xx 2,000.0 ksi

Density 45.070 pcf

Fb - Compr 2,900.0 psi

Fc - Perp 750.0 psi

Ft 2,025.0 psi

Eminbend - xx 1,016.54 ksi

Applied Loads

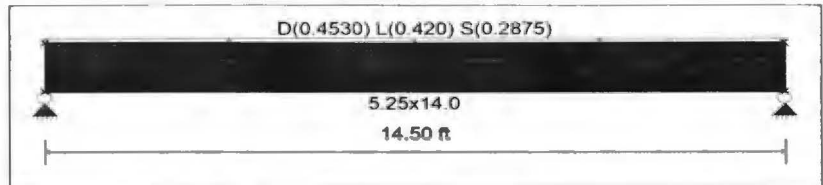
Unif Load: D = 0.4530, L = 0.420, S = 0.2875 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.563 : 1**
 fb : Actual : 1,605.38 psi at 7.250 ft in Span # 1
 Fb : Allowable : 2,850.80 psi
 Load Comb : +D+L

Max fv/FvRatio = **0.374 : 1**
 fv : Actual : 108.50 psi at 13.340 ft in Span # 1
 Fv : Allowable : 290.00 psi
 Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.28		3.05	2.08			
Right Support	3.28		3.05	2.08			



Max Deflections

Transient Downward	0.175 in	Total Downward	0.410 in
Ratio	994	Ratio	424
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:	

Wood Beam Design : 19. Floor beam over Kitchen

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

Fb - Tension 2325 psi

Fc - Prll 2050 psi

Fv 310 psi

Ebend- xx 1550 ksi

Density 45.01 pcf

Fb - Compr 2325 psi

Fc - Perp 800 psi

Ft 1070 psi

Eminbend - xx 787.815 ksi

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft

Unif Load: D = 0.1530, S = 0.0750 k/ft, 0.0 to 6.0 ft, Trib= 1.0 ft

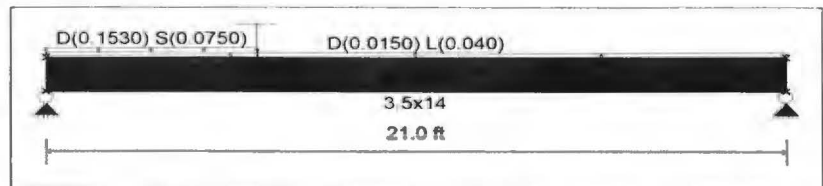
1Point: D = 0.350, L = 0.30, S = 0.2130 k @ 6.0 ft

Design Summary

Max fb/Fb Ratio = **0.331 : 1**
 fb : Actual : 758.46 psi at 6.020 ft in Span # 1
 Fb : Allowable : 2,292.26 psi
 Load Comb : +D+L

Max fv/FvRatio = **0.158 : 1**
 fv : Actual : 48.85 psi at 0.000 ft in Span # 1
 Fv : Allowable : 310.00 psi
 Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.19		0.63	0.54			
Right Support	0.39		0.51	0.13			



Max Deflections

Transient Downward	0.204 in	Total Downward	0.452 in
Ratio	1234	Ratio	557
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:	

Description :

Wood Beam Design : 20. Typical Header at Main Floor level

Calculations per NDS 2018, IBC 2018, CBC 2019

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

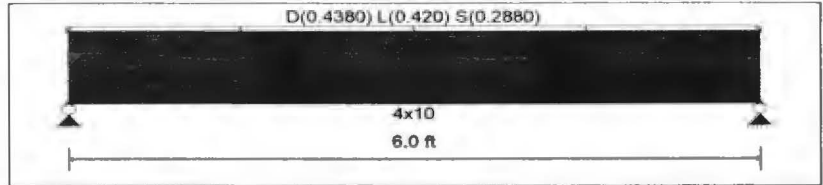
Unif Load: D = 0.4380, L = 0.420, S = 0.2880 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.868 : 1
 fb : Actual : 928.28 psi at 3.000 ft in Span # 1
 Fb : Allowable : 1,069.18 psi
 Load Comb : +D+L

Max fv/FvRatio = 0.495 : 1
 fv : Actual : 89.05 psi at 5.240 ft in Span # 1
 Fv : Allowable : 180.00 psi
 Load Comb : +D+L

Max Reactions (k) D Lr L S W E H
 Left Support 1.31 1.26 0.86
 Right Support 1.31 1.26 0.86



Max Deflections

Transient Downward 0.033 in Total Downward 0.077 in
 Ratio 2159 Ratio 936
 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:

Wood Beam Design : 21. Low beam over Storage Rm

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x9, 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 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

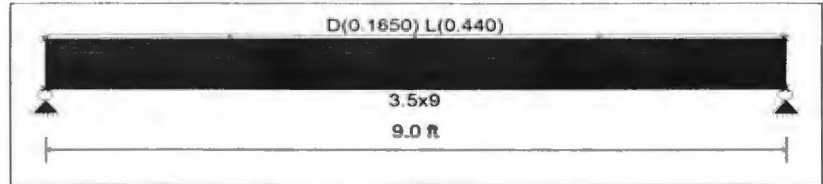
Unif Load: D = 0.1650, L = 0.440 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.648 : 1
 fb : Actual : 1,555.71 psi at 4.500 ft in Span # 1
 Fb : Allowable : 2,400.00 psi
 Load Comb : +D+L

Max fv/FvRatio = 0.411 : 1
 fv : Actual : 108.90 psi at 8.280 ft in Span # 1
 Fv : Allowable : 265.00 psi
 Load Comb : +D+L

Max Reactions (k) D Lr L S W E H
 Left Support 0.74 1.98
 Right Support 0.74 1.98



Max Deflections

Transient Downward 0.171 in Total Downward 0.235 in
 Ratio 632 Ratio 460
 LC: L Only LC: +D+L
 Transient Upward 0.000 in Total Upward 0.000 in
 Ratio 9999 Ratio 9999
 LC: LC:

Wood Beam Design : 22. Low beam over Powder Rm

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Braced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

Fb - Tension 2,325.0 psi

Fc - Prll 2,050.0 psi

Fv 310.0 psi

Ebend- xx 1,550.0 ksi

Density 45.010 pcf

Fb - Compr 2,325.0 psi

Fc - Perp 800.0 psi

Ft

1,070.0 psi Eminbend - xx 787.82 ksi

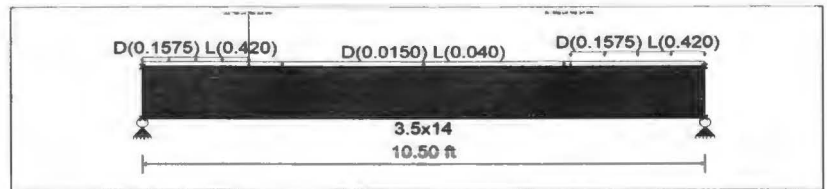
Applied Loads

- Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
- Unif Load: D = 0.1575, L = 0.420 k/ft, 0.0 to 2.0 ft, Trib= 1.0 ft
- Unif Load: D = 0.1575, L = 0.420 k/ft, 8.0 to 10.50 ft, Trib= 1.0 ft
- 1Point: D = 2.190, L = 0.780, S = 2.360 k @ 2.0 ft
- 2Point: D = 1.650, L = 0.780, S = 1.690 k @ 8.0 ft

Design Summary

Max fb/Fb Ratio = **0.428 : 1**
 fb : Actual : 1,129.12 psi at 5.670 ft in Span # 1
 Fb : Allowable : 2,636.10 psi
 Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.442 : 1**
 fv : Actual : 157.57 psi at 0.000 ft in Span # 1
 Fv : Allowable : 356.50 psi
 Load Comb : +D+0.750L+0.750S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.58		1.91	2.31			
Right Support	2.13		1.96	1.74			

Max Deflections

Transient Downward	0.081 in	Total Downward	0.194 in
Ratio	1553	Ratio	648
	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 : 23. Rim beam at Side of Lower Stair

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Unbraced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

Fb - Tension 2,325.0 psi

Fc - Prll 2,050.0 psi

Fv 310.0 psi

Ebend- xx 1,550.0 ksi

Density 45.010 pcf

Fb - Compr 2,325.0 psi

Fc - Perp 800.0 psi

Ft

1,070.0 psi Eminbend - xx 787.82 ksi

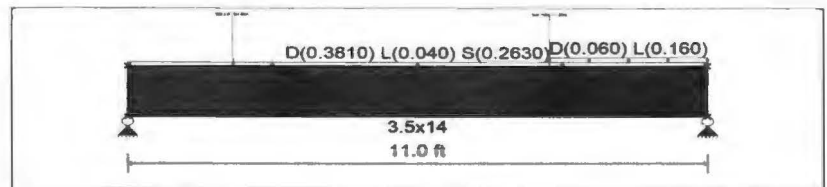
Applied Loads

- Unif Load: D = 0.3810, L = 0.040, S = 0.2630 k/ft, Trib= 1.0 ft
- Unif Load: D = 0.060, L = 0.160 k/ft, 8.0 to 11.0 ft, Trib= 1.0 ft
- 1Point: D = 0.530, S = 0.880 k @ 2.0 ft
- 2Point: D = 0.1050, L = 0.280 k @ 8.0 ft

Design Summary

Max fb/Fb Ratio = **0.517 : 1**
 fb : Actual : 1,204.37 psi at 5.170 ft in Span # 1
 Fb : Allowable : 2,329.98 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.345 : 1**
 fv : Actual : 122.96 psi at 0.000 ft in Span # 1
 Fv : Allowable : 356.50 psi
 Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.58		0.36	2.17			
Right Support	2.42		0.84	1.61			

Max Deflections

Transient Downward	0.088 in	Total Downward	0.206 in
Ratio	1497	Ratio	639
	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 : 24. Rim beam at Stair supporting beam 23

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : 3.5x14, TimberStrand LSL, Fully Unbraced

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

Wood Species : iLevel Truss Joist

Wood Grade : TimberStrand LSL 1.55E

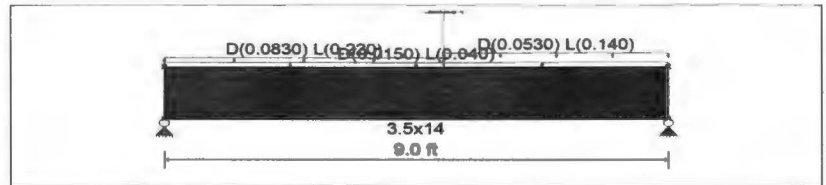
Fb - Tension 2,325.0 psi Fc - Prll 2,050.0 psi Fv 310.0 psi Ebend- xx 1,550.0 ksi Density 45.010 pcf
 Fb - Compr 2,325.0 psi Fc - Perp 800.0 psi Ft 1,070.0 psi Eminbend - xx 787.82 ksi

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
 Unif Load: D = 0.0830, L = 0.220 k/ft, 0.0 to 5.0 ft, Trib= 1.0 ft
 Unif Load: D = 0.0530, L = 0.140 k/ft, 5.0 to 9.0 ft, Trib= 1.0 ft
 1Point: D = 2.580, L = 0.360, S = 2.170 k @ 5.0 ft
 2Point: D = 0.810, S = 1.340 k @ 5.0 ft

Design Summary

Max fb/Fb Ratio = **0.714 : 1**
 fb : Actual : 1,729.27 psi at 5.010 ft in Span # 1
 Fb : Allowable : 2,422.22 psi
 Load Comb : +D+0.750L+0.750S
 Max fv/FvRatio = **0.369 : 1**
 fv : Actual : 131.72 psi at 7.860 ft in Span # 1
 Fv : Allowable : 356.50 psi
 Load Comb : +D+0.750L+0.750S



Max Reactions (k) D Lr L S W E H

Left Support	1.92		1.26	1.56			
Right Support	2.23		1.12	1.95			

Max Deflections

Transient Downward	0.073 in	Total Downward	0.162 in
Ratio	1471	Ratio	666
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 : 25. Header at Basement level

Calculations per NDS 2018, IBC 2018, CBC 2019

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

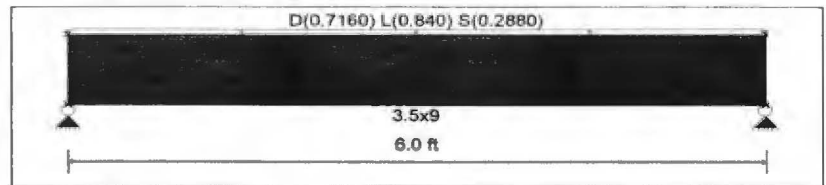
Fb - Tension 2400 psi Fc - Prll 1650 psi Fv 265 psi Ebend- xx 1800 ksi Density 31.21 pcf
 Fb - Compr 1850 psi Fc - Perp 650 psi Ft 1100 psi Eminbend - xx 950 ksi

Applied Loads

Unif Load: D = 0.7160, L = 0.840, S = 0.2880 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.749 : 1**
 fb : Actual : 1,778.29 psi at 3.000 ft in Span # 1
 Fb : Allowable : 2,375.61 psi
 Load Comb : +D+L
 Max fv/FvRatio = **0.632 : 1**
 fv : Actual : 167.46 psi at 5.260 ft in Span # 1
 Fv : Allowable : 265.00 psi
 Load Comb : +D+L



Max Reactions (k) D Lr L S W E H

Left Support	2.15		2.52	0.86			
Right Support	2.15		2.52	0.86			

Max Deflections

Transient Downward	0.064 in	Total Downward	0.120 in
Ratio	1119	Ratio	601
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:	

Wood Beam Design : 26. Gable Header

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x8, 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 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

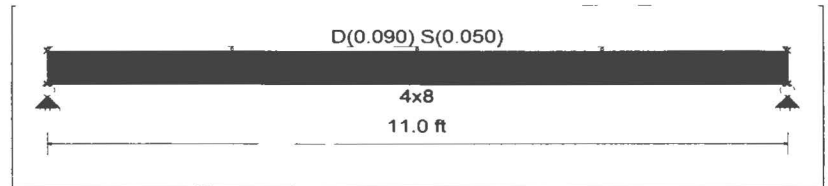
Unif Load: D = 0.090, S = 0.050 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.627 : 1**
 fb : Actual : 828.73 psi at 5.500 ft in Span # 1
 Fb : Allowable : 1,320.73 psi
 Load Comb : +D+S

Max fv/FvRatio = **0.196 : 1**
 fv : Actual : 40.66 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	0.50			0.28			
Right Support	0.50			0.28			



Max Deflections

Transient Downward	0.093 in	Total Downward	0.261 in
Ratio	1417	Ratio	506
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC: