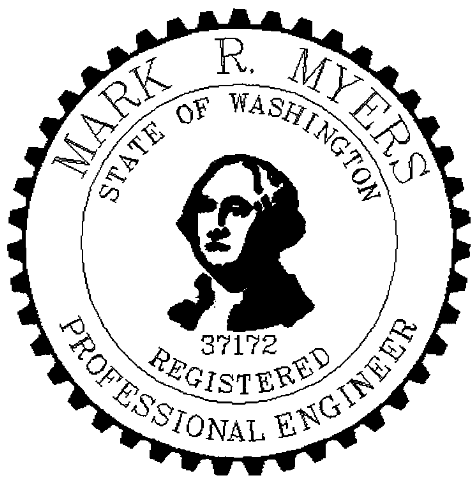
MYERS ENGINEERING

Structural Calculations



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Mark Myers, PE
Date: 2021.10.28
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MUST BEAR ORIGINAL BLUE INK SIGNATURE OR
DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: Masin Residence
7208 North Mercer Way
Mercer Island, WA

October 28, 2021

2018 INTERNATIONAL BUILDING CODE
100 MPH WIND, EXPOSURE D, K_{zt} = 1.18
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

3206 50th Street Court, Suite 210-B
Gig Harbor, WA 98335
Phone: 253-858-3248
Email: myengineer@centurytel.net

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 - 6X OR LARGER--------DF#2
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.
Fb=2,400 PSI, Fv=165 PSI, Fc (Perp) =650 PSI, E=1,800,000 PSI.

PARALLAM (PSL) 2.0E BEAM & HEADER.
Fb=2,900 PSI, Fv=290 PSI, Fc (Perp) =750 PSI, E=2,000,000 PSI.

MICROLAM (LVL) 1.9E BEAM & HEADER
Fb=2,600 PSI, Fv=285 PSI, Pc (Perp) =750 PSI, E=1,900,000 PSI.

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD
Fb=1,700 PSI, Fv=400 PSI, Pc (Perp) =680 PSI, E=1,300,000 PSI.

TRUSSES:

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

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

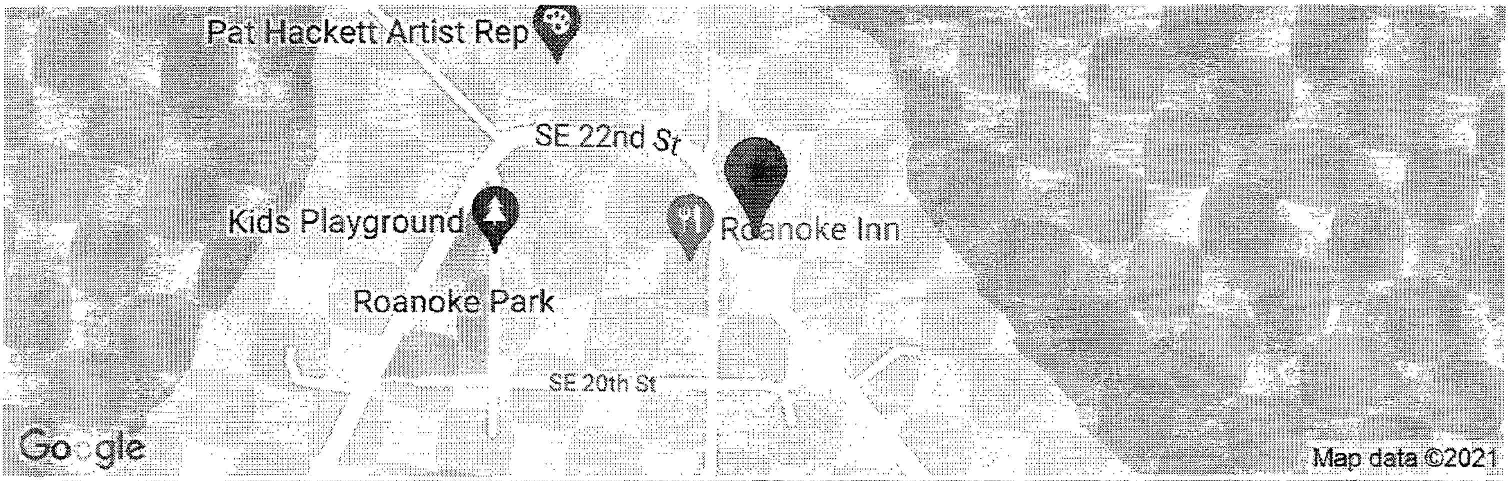
ENGINEERED I-JOISTS

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

OSHPD

Masin Residence

Latitude, Longitude: 47.5941, -122.2427



Date	10/14/2021, 5:10:31 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.382	MCE_R ground motion. (for 0.2 second period)
S_1	0.481	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.658	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.105	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.591	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.709	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.382	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.531	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.02	Factored deterministic acceleration value. (0.2 second)
S1RT	0.481	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.537	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.25	Factored deterministic acceleration value. (1.0 second)
PGAd	1.052	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.903	Mapped value of the risk coefficient at short periods
C_{R1}	0.896	Mapped value of the risk coefficient at a period of 1 s

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3206 50th Street Ct NW, Ste 210-B
Gig Harbor, WA 98335

PROJECT :**Masin Residence**

Phone: 253-858-3248
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LATERAL ANALYSIS :

BASED ON 2018 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 2018 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)

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

$S_s := 1.382$ $\quad S_1 := 0.481$ $\quad S_{ms} := 1.658$ $\quad S_{m1} := 0.866$

Equation 11.4-3 $\quad S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.11$ $\quad$ Equation 11.4-4 $\quad S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.58$

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

Roof Slope Adjustment Factor: $S_a := \dfrac{1}{\cos\left(\operatorname{atan}\left(\frac{3}{12}\right)\right)} = 1.03$

Plan Area for Each Level:

$A_1 := 2600\text{ft}^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1970\text{ft}^2$ (Upper Floor) $\quad A_{2b} := 1594\text{ft}^2 \cdot S_a$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(42.5\text{ft}) + 2(51.5\text{ft})$ (Upper Floor) $\quad P_2 := 2(56\text{ft}) + 2(78\text{ft})$ (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:

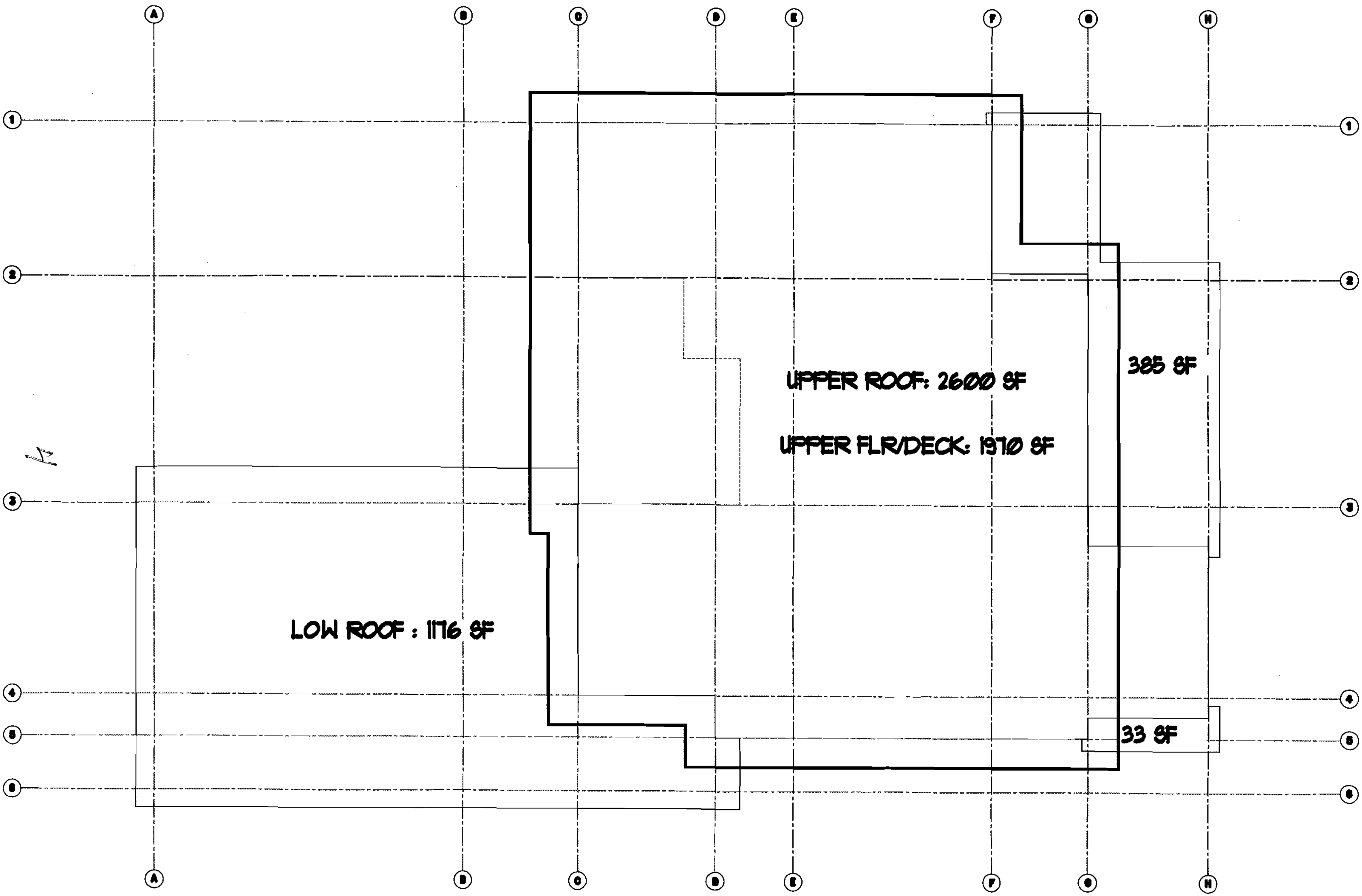
$$w_1 := 15 \cdot \text{psf} \cdot A_1 + 20 \cdot \text{psf} \cdot 4.5 \cdot \text{ft} \cdot P_1$$

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

Story Weight at Main Floor:

$$w_2 := 15 \cdot \text{psf} \cdot (A_{2a} + A_{2b}) + 20 \cdot \text{psf} \cdot (4.5 \cdot \text{ft} \cdot P_1 + 5\text{ft} \cdot P_2)$$

$$W := w_1 + w_2 = 155036.14\,\text{lb}$$


UPPER ROOF: 2600 SF
UPPER FLR/DECK: 1970 SF
385 SF
33 SF
LOW ROOF : 1176 SF

Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $\quad h_n := 25$ (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) $\quad T_L := 6$ (per ASCE 7-16 Fig. 22-14)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.4$ (ASCE 7-16 Eq. 12.8-3)

C_s shall not be less than: $0.044 S_{DS} \cdot I_e = 0.05$ (ASCE 7-16 Eq. 12.8-5)

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.17$$ (ASCE 7-16 Eq. 12.8-2)

Total Base Shear: $V_E := C_s \cdot W = 26364.1\ \text{lb}$

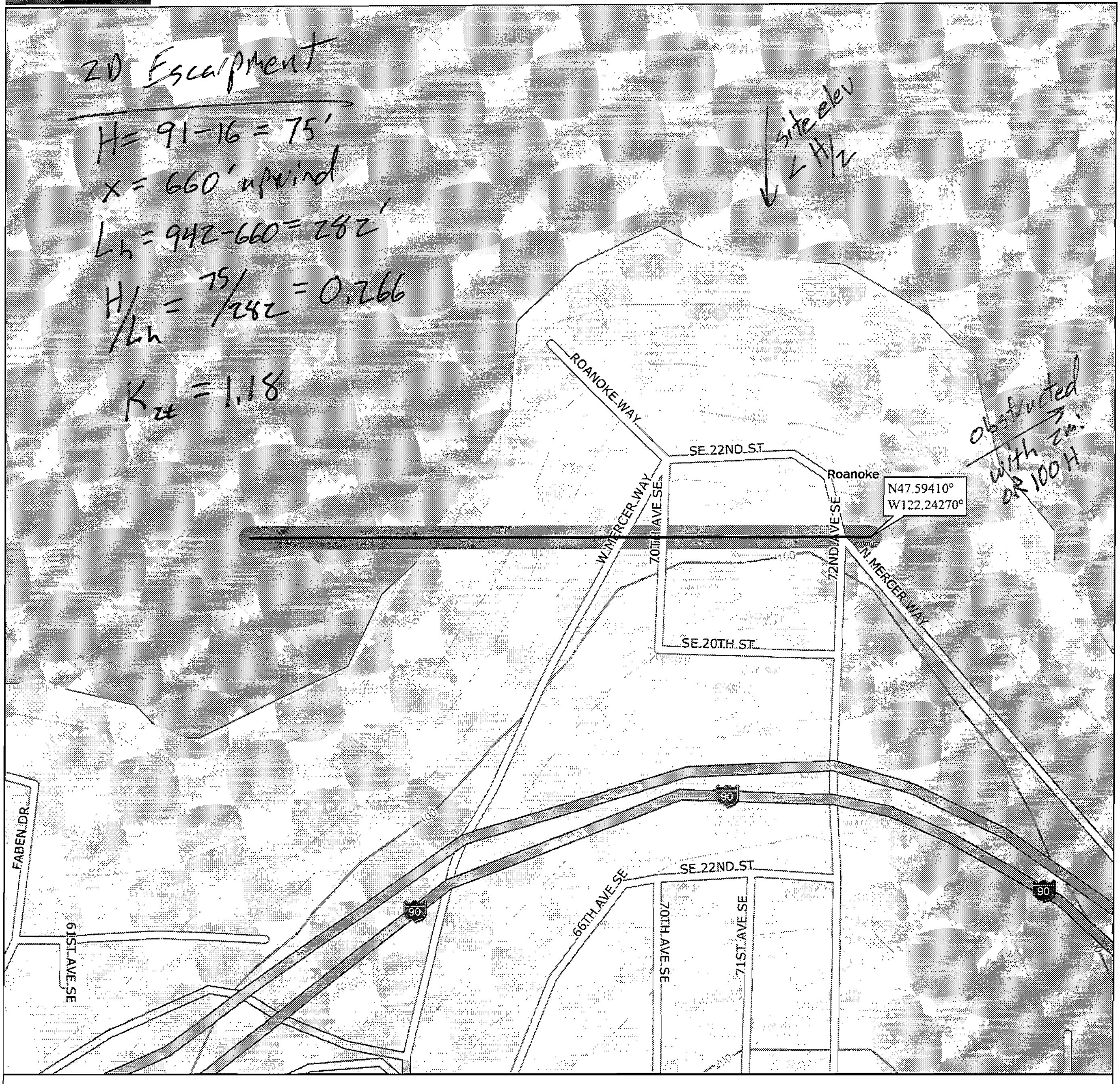
Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

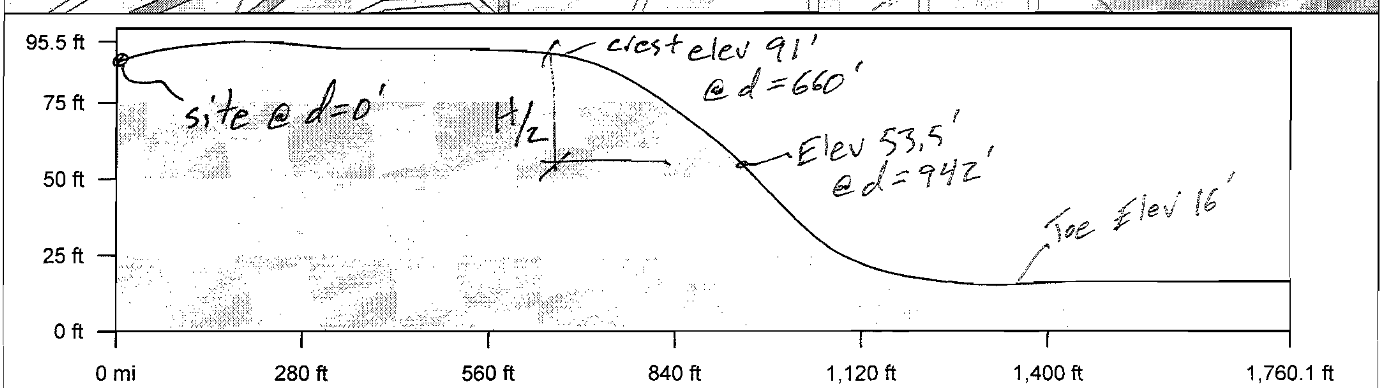
for structures having a period of 0.5 sec or less: $k := 1$

$h_1 := 20\text{ft}$ $\quad h_2 := 10\text{ft}$ (Height from base to level x)

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.54$$ $F_1 := C_{v1} \cdot V_E = 14196.36\ \text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.46$$ $F_2 := C_{v2} \cdot V_E = 12167.73\ \text{lb}$ Story Shear at Main Floor





Lin Dist: 1,753.9 ft	Terr Dist: 1,760.1 ft	Elev Gain: -72.2 ft	Avg Grade: 5
Climb Elev: 8.3 ft	Desc Elev: 80.6 ft	Max. Elev: 95.5 ft	Min. Elev: 15.3 ft
Climb Dist: 485.1 ft	Desc Dist: 1,142.7 ft		

TN
MN (14.8°E)

Scale 1 : 5,200
1" = 433.3 ft
Data Zoom 15-3

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 D (ASCE 7-16 Sect. 26.7.3)

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

$x := 660ft$ $H := 75ft$ $L_h := 282ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.95\left(\frac{H}{L_h}\right) = 0.25 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.41 \qquad K_3 := e^{\frac{(-\gamma\cdot z)}{L_h}} = 0.8 \qquad K_{zt} := \left(1 + K_1\cdot K_2\cdot K_3\right)^2 = 1.18$$

$G := 0.85$ 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):

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

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

$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 1.08 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 1.03 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 1.13$$

External Pressure Coefficients w/ Roof Pitch = 3/12 (14 degrees) Front to Back & 3/12 (14 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 42.5ft$ $B_{fb} := 51.5ft$ $\frac{L_{fb}}{B_{fb}} = 0.83$ $\frac{h}{L_{fb}} = 0.59$

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

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

$C_{pf3} := -.52$ Leeward Roof

$C_{pf4} := -.4$ Leeward Wall

Side to Side:

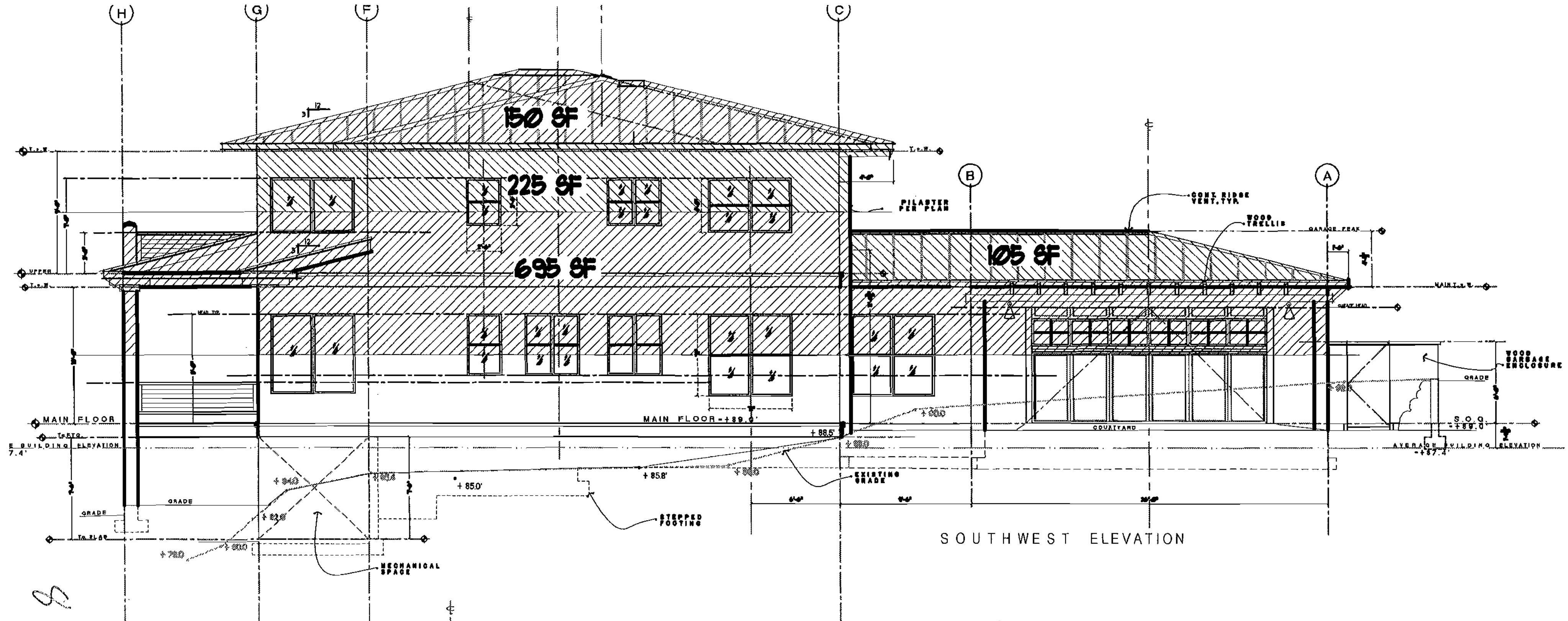
$L_{ss} := 51.5ft$ $B_{ss} := 42.5ft$ $\frac{L_{ss}}{B_{ss}} = 1.21$ $\frac{h}{L_{ss}} = 0.49$

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

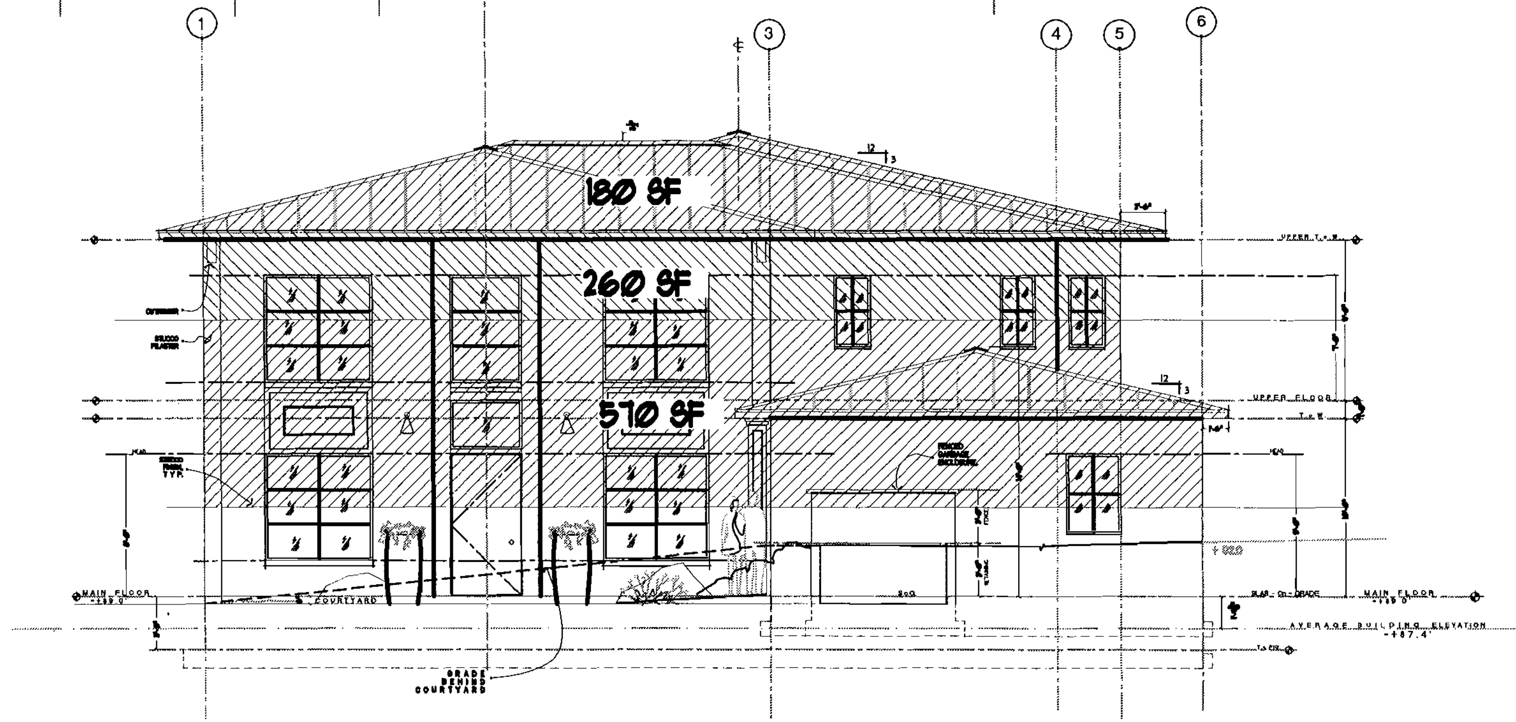
$C_{ps2} := -0.18$ Windward Roof

$C_{ps3} := -.5$ Leeward Roof

$C_{ps4} := -.46$ Leeward Wall



SOUTHWEST ELEVATION



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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 27.69$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 26.34$ $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 28.79$

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

Windward Wall Both Directions $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 18.83\,lb \cdot ft^{-2}$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 17.91\,lb \cdot ft^{-2}$

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -4.4\,lb \cdot ft^{-2}$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -12.72\,lb \cdot ft^{-2}$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -9.79\,lb \cdot ft^{-2}$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = -4.4\,lb \cdot ft^{-2}$

Leeward Roof Side to Side $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -12.24\,lb \cdot ft^{-2}$

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -11.26\,lb \cdot ft^{-2}$

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

$p_{wr1} - p_{lr1} = 8.32\,lb \cdot ft^{-2}$ $p_{ww1} - p_{lw1} = 28.62\,lb \cdot ft^{-2}$ $p_{ww2} - p_{lw1} = 27.7\,lb \cdot ft^{-2}$

$p_{wr2} - p_{lr2} = 7.83\,lb \cdot ft^{-2}$ $p_{ww1} - p_{lw2} = 30.09\,lb \cdot ft^{-2}$ $p_{ww2} - p_{lw2} = 29.17\,lb \cdot ft^{-2}$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})180ft^2 + (p_{ww1} - p_{lw1}) \cdot 260 \cdot ft^2 = 8938.66\,lb$$

Wind Pressure at Main Floor (Front to Back):

$$V_{2W} := (p_{wr1} - p_{lr1})0ft^2 + (p_{ww2} - p_{lw1}) \cdot 570 \cdot ft^2 = 15789.29\,lb$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (8psf) \cdot 150ft^2 + (p_{ww1} - p_{lw2}) \cdot 225ft^2 = 7969.74\,lb$$

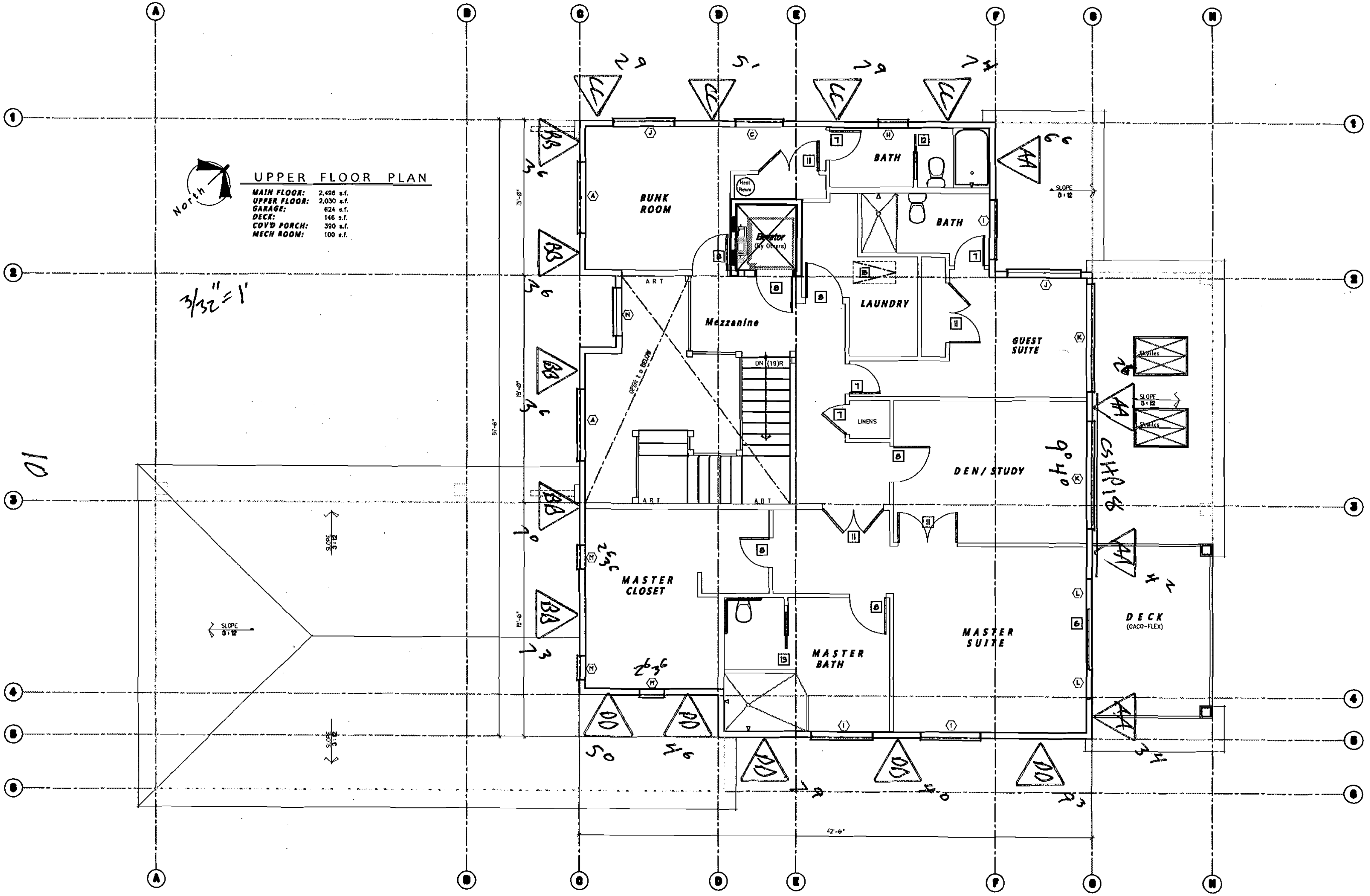
Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (8psf) \cdot 105ft^2 + (p_{ww2} - p_{lw2}) \cdot 695ft^2 = 21112.27\,lb$$

UPPER FLOOR PLAN

MAIN FLOOR:	2,496 s.f.
UPPER FLOOR:	2,030 s.f.
GARAGE:	624 s.f.
DECK:	146 s.f.
COV'D PORCH:	390 s.f.
MECH ROOM:	100 s.f.

3/32" = 1'



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PROJECT :**Masin Residence**

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WALL AA:

Story Shear due to Wind: $V_{3W} = 7969.74\,lb$ Story Shear due to Seismic: $F_1 = 14196.36\,lb$

Bldg Width in direction of Load: $L_t := 42.5\cdot ft$ Distance between shear walls: $L_1 := 42.5\cdot ft$

Shear Wall Length: $Laa := \left[6.5 + 2.5 + 4.17 + 3.33\left(\frac{6.66}{9}\right)\right]ft = 15.63\,ft$

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

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

Wind Force: $vaa := \dfrac{\frac{0.6V_{3W}}{L_t}\cdot\frac{L_1}{2}}{Laa}$

Seismic Force: $\rho := 1.0$ $E_{aa} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Laa}$

$vaa = 152.93\,lb\cdot ft^{-1}$ $\frac{vaa}{C_o} = 152.93\,lb\cdot ft^{-1}$ $E_{aa} = 317.81\,lb\cdot ft^{-1}$ $\frac{E_{aa}}{C_o} = 317.81\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_{aa} := 15.67\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{aa} := (15\cdot psf)\cdot 6.5\cdot ft + (20\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 2174.21\,lb$

Chord Force:

$CFaa_w := \frac{vaa\cdot 6.67ft\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 585.85\,lb$

$CFaa_s := \frac{E_{aa}\cdot 6.67ft\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 1217.5\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = -718.67\,lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 249.42\,lb$

No Holdown Required

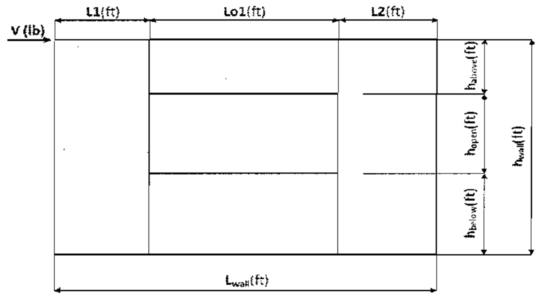
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2018 IBC	**Date:** 10/26/2021
Designer:	Mark Myers, PE	
Client:	RFA/RKK	
Project:	Masin Residence	
Wall Line:	AA	



Shear Wall Calculation Variables

V	2121 lbf		Opening 1	Adj. Factor Method =	2bs/h	
L1	2.50 ft	ha1	2.00 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	4.17 ft	ho1	4.00 ft	P1=ho1/L1=	1.60	N/A
h_{wall}	9.00 ft	hb1	3.00 ft	P2=ho2/L2=	0.96	N/A
L_{wall}	15.67 ft	Lo1	9.00 ft			

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1218 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) =	244 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	2193 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	822 lbf
F2 = O1(L2)/(L1+L2) =	1371 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.37 ft
T2 = (L2*Lo1)/(L1+L2) =	5.63 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 =	318 plf	
V2 = (V/L)(T2+L2)/L2 =	318 plf	
Check V1*L1+V2*L2=V?	2121 lbf	**OK**

7. Resistance to corner forces

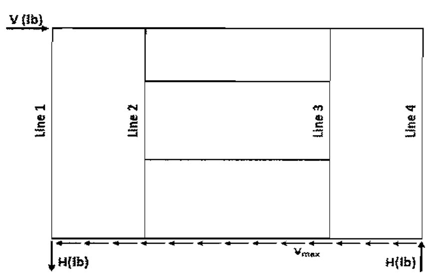
R1 = V1*L1 =	795 lbf
R2 = V2*L2 =	1326 lbf

8. Difference corner force + resistance

R1-F1 =	-27 lbf
R2-F2 =	-45 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-11 plf
vc2 = (R2-F2)/L2 =	-11 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-54	1273	1218 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1218	-54	1273	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	1218	-54	1273	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		-54	1273	1218 lbf

Design Summary*

Req. Sheathing Capacity	318 plf	**4-Term Deflection**	0.048 in.	**3-Term Deflection**	0.060 in.
Req. Strap Force	1371 lbf	**4-Term Story Drift %**	0.002 %	**3-Term Story Drift %**	0.002 %
Req. HD Force (H)	1218 lbf ✓		See Page 2		See Page 3
Req. Shear Wall Anchorage Force (v_{max})	135 plf				

CSHP18

*The Design Summary assumes that the shear wall is designed as blocked.

Dead Load Resisting Overturning: $L_{aa} := 3.33 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{aa} := (15 \cdot psf) \cdot 3 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRaa := \dfrac{W_{aa} \cdot L_{aa}}{2}$ $DLRaa = 374.63\ lb$

Chord Force:

$CFaa_w := \dfrac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_w = 1376.36\ lb$

$CFaa_s := \dfrac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_s = 2860.3\ lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 1151.58\ lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 2693.5\ lb$

Simpson MSTC40 at wall below or MSTC28 at flush/rim beam

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_p := \dfrac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 1.07\ ft$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 0.51\ ft$

16d @ 6" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\ lb$

$As := \dfrac{(Z_B \cdot C_o)}{vaa} = 9\ ft$ $\dfrac{(Z_B \cdot C_o)}{E_{aa}} = 4.33\ ft$

5/8" A.B. @ 48" o.c.

WALL BB:

Story Shear due to Wind: $V_{3W} = 7969.74\,lb$

Story Shear due to Seismic: $F_1 = 14196.36\,lb$

Bldg Width in direction of Load: $L_t := 42.5 \cdot ft$

Distance between shear walls: $L_1 := 42.5 \cdot ft$

Shear Wall Length: $Lbb := \left[3 \cdot 3.5 \left(\frac{7}{9}\right) + 7 + 7.25\right] ft = 22.42\,ft$

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

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

Wind Force: $vbb := \dfrac{\frac{0.6V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Lbb}$

Seismic Force: $\rho := 1.0$ $\quad E_{bb} := \dfrac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1}{2}}{Lbb}$

$vbb = 106.66\,lb \cdot ft^{-1}$ $\quad \dfrac{vbb}{C_o} = 106.66\,lb \cdot ft^{-1}$

$E_{bb} = 221.65\,lb \cdot ft^{-1}$ $\quad \dfrac{E_{bb}}{C_o} = 221.65\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_{bb} := 3.5 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{bb} := (15 \cdot psf) \cdot 3 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRbb := \dfrac{W_{bb} \cdot L_{bb}}{2}$ $\quad DLRbb = 393.75\,lb$

Chord Force:

$CFbb_w := \dfrac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\quad CFbb_w = 959.92\,lb$

$CFbb_s := \dfrac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\quad CFbb_s = 1994.88\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 723.67\,lb$

$HDFbb_s := CFbb_s - \left(0.6 - 0.14S_{DS}\right) \cdot DLRbb = 1819.56\,lb$

Simpson MSTC40 at wall below or MSTC28 at flush/rim beam

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

$Z_N := 102 \cdot lb$ $\quad C_D := 1.6$

$B_R := \dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{vbb} = 1.53\,ft$ $\quad \dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_{bb}} = 0.74\,ft$

16d @ 8" o.c.

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

$A_s := 860 \cdot lb$ $\quad C_D := 1.6$ $\quad Z_B := A_s \cdot C_D$ $\quad Z_B = 1376\,lb$

$As := \dfrac{\left(Z_B \cdot C_o\right)}{vbb} = 12.9\,ft$ $\quad \dfrac{\left(Z_B \cdot C_o\right)}{E_{bb}} = 6.21\,ft$

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

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WALL CC:

Story Shear due to Wind: $V_{1W} = 8938.66\,lb$ Story Shear due to Seismic: $F_1 = 14196.36\,lb$

Bldg Width in direction of Load: $L_t := 51.5 \cdot ft$ Distance between shear walls: $L_1 := 51.5 \cdot ft$

Shear Wall Length: $Lcc := \left[2.75\left(\frac{5.5}{9}\right) + 5 + 7.75 + 7.33\right]ft = 21.76\,ft$

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

Wind Force: $vcc := \dfrac{\frac{0.6V_{1W}}{L_t} \cdot \frac{L_1}{2}}{Lcc}$ Seismic Force: $\rho := 1.0$ $E_{cc} := \dfrac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1}{2}}{Lcc}$

$vcc = 123.23\,lb \cdot ft^{-1}$ $\frac{vcc}{C_o} = 123.23\,lb \cdot ft^{-1}$ $E_{cc} = 228.34\,lb \cdot ft^{-1}$ $\frac{E_{cc}}{C_o} = 228.34\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_{cc} := 2.75 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{cc} := (15 \cdot psf) \cdot 3 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRcc := \frac{W_{cc} \cdot L_{cc}}{2}$ $DLRcc = 309.37\,lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_w = 1109.09\,lb$ $CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_s = 2055.03\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 923.46\,lb$ $HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 1917.28\,lb$

Simpson MSTC40 at wall below or MSTC28 at flush/rim beam

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 1.32\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 0.71\,ft$

16d @ 8" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vcc} = 11.17\,ft$ $\frac{(Z_B \cdot C_o)}{E_{cc}} = 6.03\,ft$

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

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WALL DD:

Story Shear due to Wind: $V_{1W} = 8938.66\,lb$ Story Shear due to Seismic: $F_1 = 14196.36\,lb$

Bldg Width in direction of Load: $L_t := 51.5 \cdot ft$ Distance between shear walls: $L_1 := 51.5 \cdot ft$

Shear Wall Length: $Ldd := \left[9.25 + 4\left(\frac{8}{9}\right) + 7.75 + 4.5 + 5\right]ft = 30.06\,ft$

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

Wind Force: $vdd := \frac{\frac{0.6V_{1W}}{L_t} \cdot \frac{L_1}{2}}{Ldd}$ Seismic Force: $\rho := 1.0$ $E_{dd} := \frac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1}{2}}{Ldd}$

$vdd = 89.22\,lb \cdot ft^{-1}$ $\frac{vdd}{C_o} = 89.22\,lb \cdot ft^{-1}$ $E_{dd} = 165.32\,lb \cdot ft^{-1}$ $\frac{E_{dd}}{C_o} = 165.32\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_{dd} := 4 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{dd} := (15 \cdot psf) \cdot 6.5 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRdd := \frac{W_{dd} \cdot L_{dd}}{2}$ $DLRdd = 555\,lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_w = 802.99\,lb$ $CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_s = 1487.86\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 469.99\,lb$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 1240.75\,lb$

Simpson MSTC40 at wall below or MSTC28 at flush/rim beam

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 1.83\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 0.99\,ft$

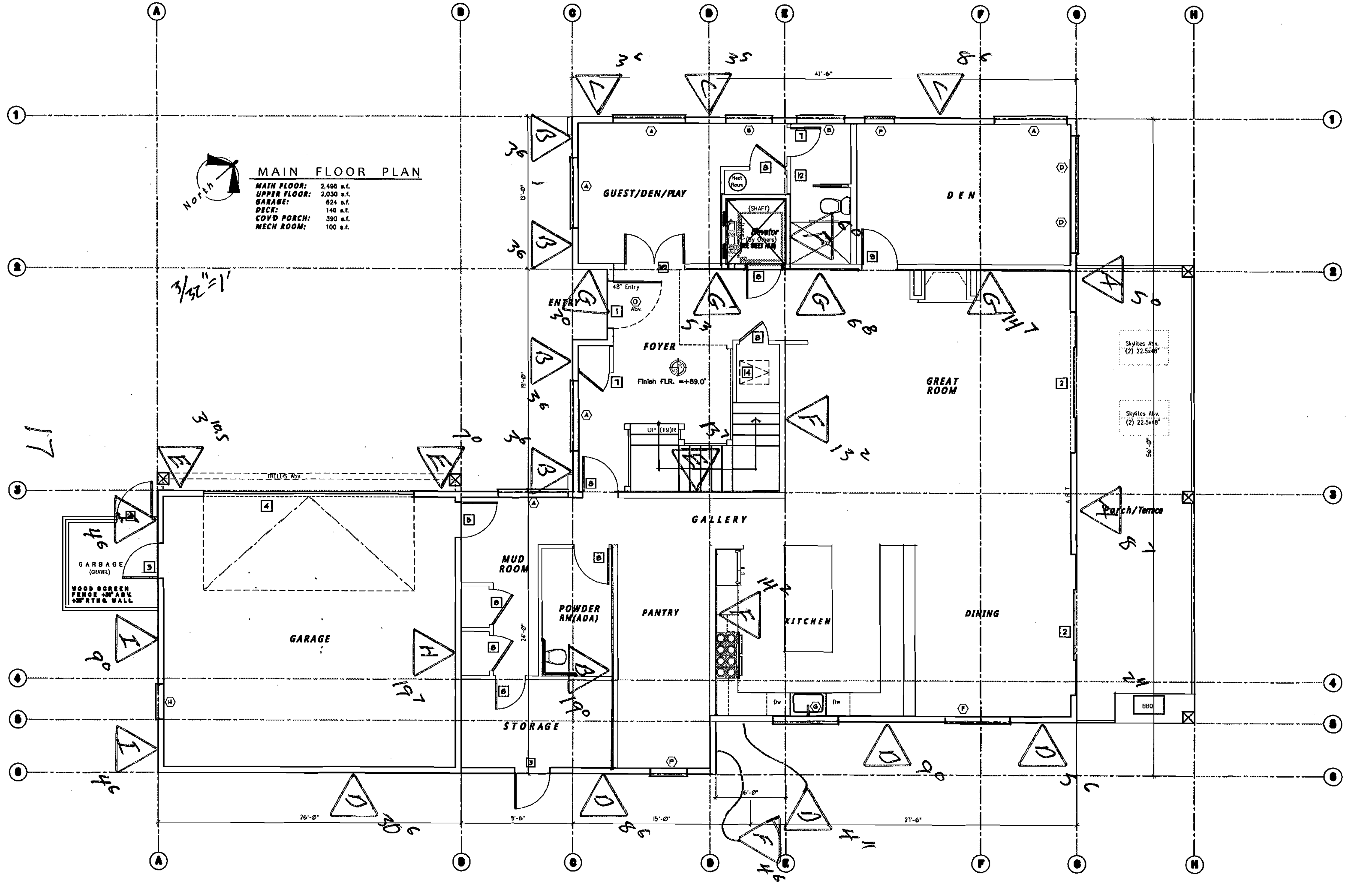
16d @ 12" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vdd} = 15.42\,ft$ $\frac{(Z_B \cdot C_o)}{E_{dd}} = 8.32\,ft$

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



MAIN FLOOR PLAN
MAIN FLOOR: 2,496 s.f.
UPPER FLOOR: 2,030 s.f.
GARAGE: 624 s.f.
DECK: 146 s.f.
COV'D PORCH: 390 s.f.
MECH ROOM: 100 s.f.
North
3/32"=1'
GUEST/DEN/PLAY
DEN
ENTRY
48" Entry
FOYER
Finish FLR. =+89.0'
GREAT ROOM
GALLERY
MUD ROOM
POWDER RM(ADA)
PANTRY
KITCHEN
DINING
STORAGE
GARAGE
GARBAGE (GRAVEL)
WOOD SCREEN FENCE +36" ABV. +36" RTNG WALL
TRELLIS Abv
Porch/Terrace
Skylites Abv. (2) 22.5x48"
Skylites Abv. (2) 22.5x48"
BBQ
(SHAFT)
Elevator (By Others)
Mech Room
UP (19)R
41'-6"
13'-0"
19'-0"
26'-0"
9'-6"
15'-0"
27'-6"
24'-0"
56'-0"
6'-0"
17

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WALL A:

Story Shear due to Wind: $V_{4W} = 21112.27\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 78 \cdot ft$ Distance between shear walls: $L_1 := 27.5 \cdot ft$

Shear Wall Length: $La := (5 + 8.583)ft = 13.58\,ft$

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

Wind Force: $va := \dfrac{vaa \cdot Laa + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{La}$ Seismic Force: $\rho := 1.0$ $E_a := \dfrac{E_{aa} \cdot Laa + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$va = 340.42\,lb \cdot ft^{-1}$ $\frac{va}{C_o} = 340.42\,lb \cdot ft^{-1}$ $E_a = 476.34\,lb \cdot ft^{-1}$ $\frac{E_a}{C_o} = 476.34\,lb \cdot ft^{-1}$

P1-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 686 plf
Seismic Capacity = 490 plf

Dead Load Resisting Overturning: $L_a := 5 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_a := (15 \cdot psf) \cdot 5 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRa := \frac{W_a \cdot L_a}{2}$ $DLRa = 712.5\,lb$

Chord Force:

$CFa_w := \dfrac{va \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_w = 3404.22\,lb$ $CFa_w + CFaa_w = 4780.58\,lb$

$CFa_s := \dfrac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_s = 4763.45\,lb$ $CFa_s + CFaa_s = 7623.75\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 2976.72\,lb$

$HDFa_w + HDFaa_w = 4128.3\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS}) \cdot DLRa = 4446.21\,lb$

$HDFa_s + HDFaa_s = 7139.71\,lb$

Simpson HDU8 at 4.5"x5.5" post

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \dfrac{(C_D \cdot Z_N \cdot C_o)}{va} = 0.48\,ft$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.34\,ft$

16d @ 4" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \dfrac{(Z_B \cdot C_o)}{va} = 4.04\,ft$ $\dfrac{(Z_B \cdot C_o)}{E_a} = 2.89\,ft$

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

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WALL B:

Story Shear due to Wind: $V_{4W} = 21112.27\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 78 \cdot ft$ Distance between shear walls: $L_1 := 15 \cdot ft$ $L_2 := 9.5ft$

Shear Wall Length: $Lb := \left[2 \cdot 3.5 \left(\frac{7}{10}\right) + 19\right] ft = 23.9\,ft$

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

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

Wind Force: $vb := \dfrac{vbb \cdot Lbb + \left(\frac{0.6 V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lb}$

Seismic Force: $\rho := 1.0$ $E_b := \dfrac{E_{bb} \cdot Lbb + \left(\rho \cdot \frac{0.7 F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lb}$

$vb = 183.28\,lb \cdot ft^{-1}$ $\frac{vb}{C_o} = 183.28\,lb \cdot ft^{-1}$

$E_b = 263.87\,lb \cdot ft^{-1}$ $\frac{E_b}{C_o} = 263.87\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 364 plf
Seismic Capacity = 260 plf (within 2%)

Dead Load Resisting Overturning: $L_b := 19 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$

$DLRb := \dfrac{W_b \cdot L_b}{2}$ $DLRb = 1045\,lb$

Chord Force:

$CFb_w := \dfrac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_w = 1832.78\,lb$

$CFb_s := \dfrac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_s = 2638.66\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1205.78\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14 S_{DS}) \cdot DLRb = 2173.37\,lb$

Simpson MSTC40 to cripple wall & HDU2 w/ SSTB16 or PAB5 anchor (6" embed) to foundation

Dead Load Resisting Overturning: $L_b := 3.5 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (20 \cdot psf) \cdot Pt + (10psf) \cdot 1 ft$

$DLRb := \dfrac{W_b \cdot L_b}{2}$ $DLRb = 367.5 \, lb$

Chord Force:

$CFb_w := \dfrac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_w = 1832.78 \, lb$

$CFb_s := \dfrac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_s = 2638.66 \, lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1612.28 \, lb$

$HDFb_s := CFb_s - \left(0.6 - 0.14 S_{DS}\right) \cdot DLRb = 2475.03 \, lb$

$HDFb_w + HDFbb_w = 2335.95 \, lb$

$HDFb_s + HDFbb_s = 4294.59 \, lb$

Simpson STHD14/RJ (within 1%) or 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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{vb} = 0.89 \, ft$ $\dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_b} = 0.62 \, ft$

16d @ 8" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376 \, lb$

$As := \dfrac{\left(Z_B \cdot C_o\right)}{vb} = 7.51 \, ft$ $\dfrac{\left(Z_B \cdot C_o\right)}{E_b} = 5.21 \, ft$

5/8" A.B. @ 60" o.c.

WALL C:

Story Shear due to Wind: $V_{2W} = 15789.29\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 56\cdot ft$ Distance between shear walls: $L_1 := 13\cdot ft$

Shear Wall Length: $Lc := \left[3.5\left(\frac{7}{10}\right) + 3.42\left(\frac{6.83}{10}\right) + 8.5\right]ft = 13.29\,ft$

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

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

Wind Force: $vc := \frac{vcc\cdot Lcc + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lc}$ Seismic Force: $\rho := 1.0$ $E_c := \frac{E_{cc}\cdot Lcc + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Lc}$

$vc = 284.6\,lb\cdot ft^{-1}$ $\frac{vc}{C_o} = 284.6\,lb\cdot ft^{-1}$ $E_c = 448.4\,lb\cdot ft^{-1}$ $\frac{E_c}{C_o} = 448.4\,lb\cdot ft^{-1}$

P1-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 686 plf
Seismic Capacity = 490 plf

Dead Load Resisting Overturning: $L_c := 3.42\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_c := (15\cdot psf)\cdot 0\cdot ft + (20\cdot psf)\cdot Pt + (10psf)\cdot 6.5ft$ $DLRc := \frac{W_c\cdot L_c}{2}$ $DLRc = 453.15\,lb$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_w = 2846.04\,lb$ $CFc_w + CFcc_w = 3955.13\,lb$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_s = 4483.98\,lb$ $CFc_s + CFcc_s = 6539.01\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 2574.15\,lb$

$HDFc_w + HDFcc_w = 3497.61\,lb$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 4282.21\,lb$

$HDFc_s + HDFcc_s = 6199.49\,lb$

Simpson HDU8 w/ SB7/8x24 anchor

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

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_P := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 0.57\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 0.36\,ft$

16d @ 4" o.c.

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

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vc} = 4.83\,ft$ $\frac{(Z_B\cdot C_o)}{E_c} = 3.07\,ft$

5/8" A.B. @ 36" o.c.

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WALL D:

Story Shear due to Wind: $V_{2W} = 15789.29\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 56\cdot ft$ Distance between shear walls: $L_1 := 24\cdot ft$

Shear Wall Length: $Ld := \left[5.5 + 9 + 4.92\left(\frac{9.83}{10}\right) + 8.5 + 30.5\right]ft = 58.34\,ft$

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

Wind Force: $vd := \frac{vdd\cdot Ldd + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Ld}$ Seismic Force: $\rho := 1.0$ $E_d := \frac{E_{dd}\cdot Ldd + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Ld}$

$vd = 80.77\,lb\cdot ft^{-1}$ $\frac{vd}{C_o} = 80.77\,lb\cdot ft^{-1}$ $E_d = 116.46\,lb\cdot ft^{-1}$ $\frac{E_d}{C_o} = 116.46\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_d := 4.92\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_d := (15\cdot psf)\cdot 0\cdot ft + (20\cdot psf)\cdot Pt + (10psf)\cdot 9.5ft$ $DLRd := \frac{W_d\cdot L_d}{2}$ $DLRd = 725.7\,lb$

Chord Force:

$CFd_w := \frac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_w = 807.67\,lb$ $CFd_s := \frac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_s = 1164.61\,lb$

$CFd_w + CFdd_w = 1610.66\,lb$ $CFd_s + CFdd_s = 2652.47\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 372.25\,lb$ $HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 841.49\,lb$

No Holdown Required

$HDFd_w + HDFdd_w = 842.24\,lb$ $HDFd_s + HDFdd_s = 2082.23\,lb$

Simpson STHD10/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

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vd} = 2.02\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 1.4\,ft$

16d @ 16" o.c.

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

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vd} = 17.04\,ft$ $\frac{(Z_B\cdot C_o)}{E_d} = 11.82\,ft$

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

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PROJECT :**Masin Residence**

WALL E:

Story Shear due to Wind: $V_{2W} = 15789.29\,\text{lb}$

Story Shear due to Seismic: $F_2 = 12167.73\,\text{lb}$

Bldg Width in direction of Load: $L_t := 56\cdot\text{ft}$

Distance between shear walls: $L_1 := 24\cdot\text{ft}$ $L_2 := 19\text{ft}$

Shear Wall Length: $Le := \left[3.875\left(\frac{7.75}{10}\right) + 7 + 13.583\right]\text{ft} = 23.59\,\text{ft}$

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

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

Wind Force: $ve := \dfrac{\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Le}$

Seismic Force: $\rho := 1.0$ $E_e := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Le}$

$ve = 154.21\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{ve}{C_o} = 154.21\,\text{lb}\cdot\text{ft}^{-1}$

$E_e = 138.64\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{E_e}{C_o} = 138.64\,\text{lb}\cdot\text{ft}^{-1}$

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

Dead Load Resisting Overturning: $L_e := 3.875\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_e := (15\cdot\text{psf})\cdot 4\cdot\text{ft} + (20\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$

$DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 503.75\,\text{lb}$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 1542.08\,\text{lb}$

$CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 1386.44\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 1239.83\,\text{lb}$

$HDFe_s := CFe_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRe = 1162.15\,\text{lb}$

Simpson LSTHD8/RJ or HDU2 w/ SSTB16 Anchor

Dead Load Resisting Overturning: $L_e := 13.583 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_e := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 9.5ft$

$DLRe := \frac{W_e \cdot L_e}{2}$ $DLRe = 1324.34\,lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_w = 1542.08\,lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_s = 1386.44\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 747.48\,lb$

$HDFe_s := CFe_s - (0.6 - 0.14S_{DS}) \cdot DLRe = 796.77\,lb$

No Holdown Required

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{ve} = 1.06\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 1.18\,ft$

16d @ 12" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{ve} = 8.92\,ft$ $\frac{(Z_B \cdot C_o)}{E_e} = 9.92\,ft$

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

WALL F:

Story Shear due to Wind: $V_{4W} = 21112.27\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 78 \cdot ft$ Distance between shear walls: $L_1 := 27.5 \cdot ft$ $L_2 := 15ft$

Shear Wall Length: $Lf := (6 + 13.167 + 14.167)ft = 33.33\,ft$

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

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

Wind Force: $vf := \dfrac{\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf}$

Seismic Force: $\rho := 1.0$ $E_f := \dfrac{\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf}$

$vf = 103.53\,lb \cdot ft^{-1}$ $\frac{vf}{C_o} = 103.53\,lb \cdot ft^{-1}$ $E_f = 69.61\,lb \cdot ft^{-1}$ $\frac{E_f}{C_o} = 69.61\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_f := 6 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 330\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 1035.29\,lb$ $CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 696.12\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 837.29\,lb$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 549.19\,lb$

No Holdown Required

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

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 1.58\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 2.34\,ft$

16d @ 16" o.c.

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

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vf} = 13.29\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 19.77\,ft$

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

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PROJECT :**Masin Residence**

WALL G:

Story Shear due to Wind: $V_{2W} = 15789.29\,lb$

Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 56\cdot ft$

Distance between shear walls: $L_1 := 13\cdot ft$ $L_2 := 19ft$

Shear Wall Length: $Lg := \left[14.583 + 6.667 + 5.25 + 3\left(\frac{6}{10}\right)\right]ft = 28.3\,ft$

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

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

Wind Force: $vg := \dfrac{\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lg}$

Seismic Force: $\rho := 1.0$ $E_g := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lg}$

$vg = 95.64\,lb\cdot ft^{-1}$ $\frac{vg}{C_o} = 95.64\,lb\cdot ft^{-1}$

$E_g = 85.99\,lb\cdot ft^{-1}$ $\frac{E_g}{C_o} = 85.99\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_g := 3\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_g := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 6.5ft$

$DLRg := \frac{W_g\cdot L_g}{2}$ $DLRg = 247.5\,lb$

Chord Force:

$CFg_w := \frac{vg\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_w = 956.44\,lb$

$CFg_s := \frac{E_g\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_s = 859.91\,lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6\cdot DLRg = 807.94\,lb$

$HDFg_s := CFg_s - (0.6 - 0.14S_{DS})\cdot DLRg = 749.71\,lb$

No Holdown Required

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

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vg} = 1.71\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_g} = 1.9\,ft$

16d @ 16" o.c.

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

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$A_s := \frac{(Z_B\cdot C_o)}{vg} = 14.39\,ft$ $\frac{(Z_B\cdot C_o)}{E_g} = 16\,ft$

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

WALL H:

Story Shear due to Wind: $V_{4W} = 21112.27\,lb$ Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 78\cdot ft$ Distance between shear walls: $L_1 := 9.5\cdot ft$ $L_2 := 26ft$

Shear Wall Length: $Lh := (19.583)ft = 19.58\,ft$

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

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

Wind Force: $vh := \frac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lh}$ Seismic Force: $\rho := 1.0$ $E_h := \frac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lh}$

$vh = 147.2\,lb\cdot ft^{-1}$ $\frac{vh}{C_o} = 147.2\,lb\cdot ft^{-1}$ $E_h = 98.98\,lb\cdot ft^{-1}$ $\frac{E_h}{C_o} = 98.98\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_h := 19.583\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_h := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRh := \frac{W_h\cdot L_h}{2}$ $DLRh = 1272.89\,lb$

Chord Force:

$CFh_w := \frac{vh\cdot L_h\cdot Pt}{C_o\cdot L_h}$ $CFh_w = 1472.01\,lb$ $CFh_s := \frac{E_h\cdot L_h\cdot Pt}{C_o\cdot L_h}$ $CFh_s = 989.77\,lb$

Holdown Force:

$HDFh_w := CFh_w - 0.6\cdot DLRh = 708.27\,lb$ $HDFh_s := CFh_s - (0.6 - 0.14S_{DS})\cdot DLRh = 423\,lb$

No Holdown Required

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

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vh} = 1.11\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_h} = 1.65\,ft$

16d @ 16" o.c.

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

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vh} = 9.35\,ft$ $\frac{(Z_B\cdot C_o)}{E_h} = 13.9\,ft$

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

WALL I:

Story Shear due to Wind: $V_{4W} = 21112.27\,lb$

Story Shear due to Seismic: $F_2 = 12167.73\,lb$

Bldg Width in direction of Load: $L_t := 78\cdot ft$

Distance between shear walls: $L_1 := 26\cdot ft$

Shear Wall Length: $Li := \left[2\cdot 4.5\left(\frac{9}{10}\right) + 9\right]ft = 17.1\,ft$

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

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

Wind Force: $vi := \dfrac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}}{Li}$

Seismic Force: $\rho := 1.0$ $\quad E_i := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}}{Li}$

$vi = 123.46\,lb\cdot ft^{-1}$ $\quad \frac{vi}{C_o} = 123.46\,lb\cdot ft^{-1}$

$E_i = 83.02\,lb\cdot ft^{-1}$ $\quad \frac{E_i}{C_o} = 83.02\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_i := 4.5\cdot ft$ $\quad$ Plate Height: $Pt := 10\cdot ft$

$W_i := (15\cdot psf)\cdot 5.5\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRi := \frac{W_i\cdot L_i}{2}$ $\quad DLRi = 410.63\,lb$

Chord Force:

$CFi_w := \frac{vi\cdot L_i\cdot Pt}{C_o\cdot L_i}$ $\quad CFi_w = 1234.64\,lb$

$CFi_s := \frac{E_i\cdot L_i\cdot Pt}{C_o\cdot L_i}$ $\quad CFi_s = 830.16\,lb$

Holdown Force:

$HDFi_w := CFi_w - 0.6\cdot DLRi = 988.26\,lb$

$HDFi_s := CFi_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRi = 647.33\,lb$

No Holdown Required

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

$Z_N := 102\cdot lb$ $\quad C_D := 1.6$

$B_p := \frac{(C_D\cdot Z_N\cdot C_o)}{vi} = 1.32\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_i} = 1.97\,ft$

16d @ 16" o.c.

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

$A_s := 860\cdot lb$ $\quad C_D := 1.6$ $\quad Z_B := A_s\cdot C_D$ $\quad Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vi} = 11.14\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_i} = 16.58\,ft$

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

Diapragm Shear Check:

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

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

Unblocked Diapraghm Case 2-6 Wind Capacity = 237 plf & Seismic Capacity = 170 plf

Wall Lines AA:

$vaa \cdot \frac{Laa}{51.5ft} = 46.43\,lb \cdot ft^{-1}$ $\quad E_{aa} \cdot \frac{Laa}{51.5ft} = 96.48\,lb \cdot ft^{-1}$

Wall Lines BB:

$vbb \cdot \frac{Lbb}{48ft} = 49.81\,lb \cdot ft^{-1}$ $\quad E_{bb} \cdot \frac{Lbb}{48ft} = 103.52\,lb \cdot ft^{-1}$

Wall Lines CC:

$vcc \cdot \frac{Lcc}{37ft} = 72.48\,lb \cdot ft^{-1}$ $\quad E_{cc} \cdot \frac{Lcc}{37ft} = 134.29\,lb \cdot ft^{-1}$

Wall Lines DD:

$vdd \cdot \frac{Ldd}{42.5ft} = 63.1\,lb \cdot ft^{-1}$ $\quad E_{dd} \cdot \frac{Ldd}{42.5ft} = 116.91\,lb \cdot ft^{-1}$

Wall Lines A:

$\frac{va \cdot La - vaa \cdot Laa}{51.5ft} = 43.36\,lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La - E_{aa} \cdot Laa}{51.5ft} = 29.15\,lb \cdot ft^{-1}$ $\quad \frac{va \cdot La}{51.5ft} = 89.79\,lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La}{51.5ft} = 125.63\,lb \cdot ft^{-1}$

Wall Lines B:

$\frac{vb \cdot Lb - vbb \cdot Lbb}{24ft} = 82.89\,lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{24ft} = 55.74\,lb \cdot ft^{-1}$ $\quad \frac{vb \cdot Lb}{37ft} = 118.39\,lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb}{37ft} = 170.44\,lb \cdot ft^{-1}$

Wall Lines C:

$\frac{vc \cdot Lc - vcc \cdot Lcc}{42.5ft} = 25.87\,lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc - E_{cc} \cdot Lcc}{42.5ft} = 23.26\,lb \cdot ft^{-1}$ $\quad \frac{vc \cdot Lc}{42.5ft} = 88.97\,lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc}{42.5ft} = 140.17\,lb \cdot ft^{-1}$

Wall Lines D:

$\frac{vd \cdot Ld - vdd \cdot Ldd}{78ft} = 26.03\,lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{78ft} = 23.4\,lb \cdot ft^{-1}$ $\quad \frac{vd \cdot Ld}{78ft} = 60.41\,lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld}{78ft} = 87.1\,lb \cdot ft^{-1}$

Wall Line E:

$\frac{ve \cdot Le}{78ft} = 46.63\,lb \cdot ft^{-1}$ $\quad \frac{E_e \cdot Le}{78ft} = 41.92\,lb \cdot ft^{-1}$

Wall Line F:

$\frac{vf \cdot Lf}{51.5ft} = 67.01\,lb \cdot ft^{-1}$ $\quad \frac{E_f \cdot Lf}{51.5ft} = 45.06\,lb \cdot ft^{-1}$

Wall Line G:

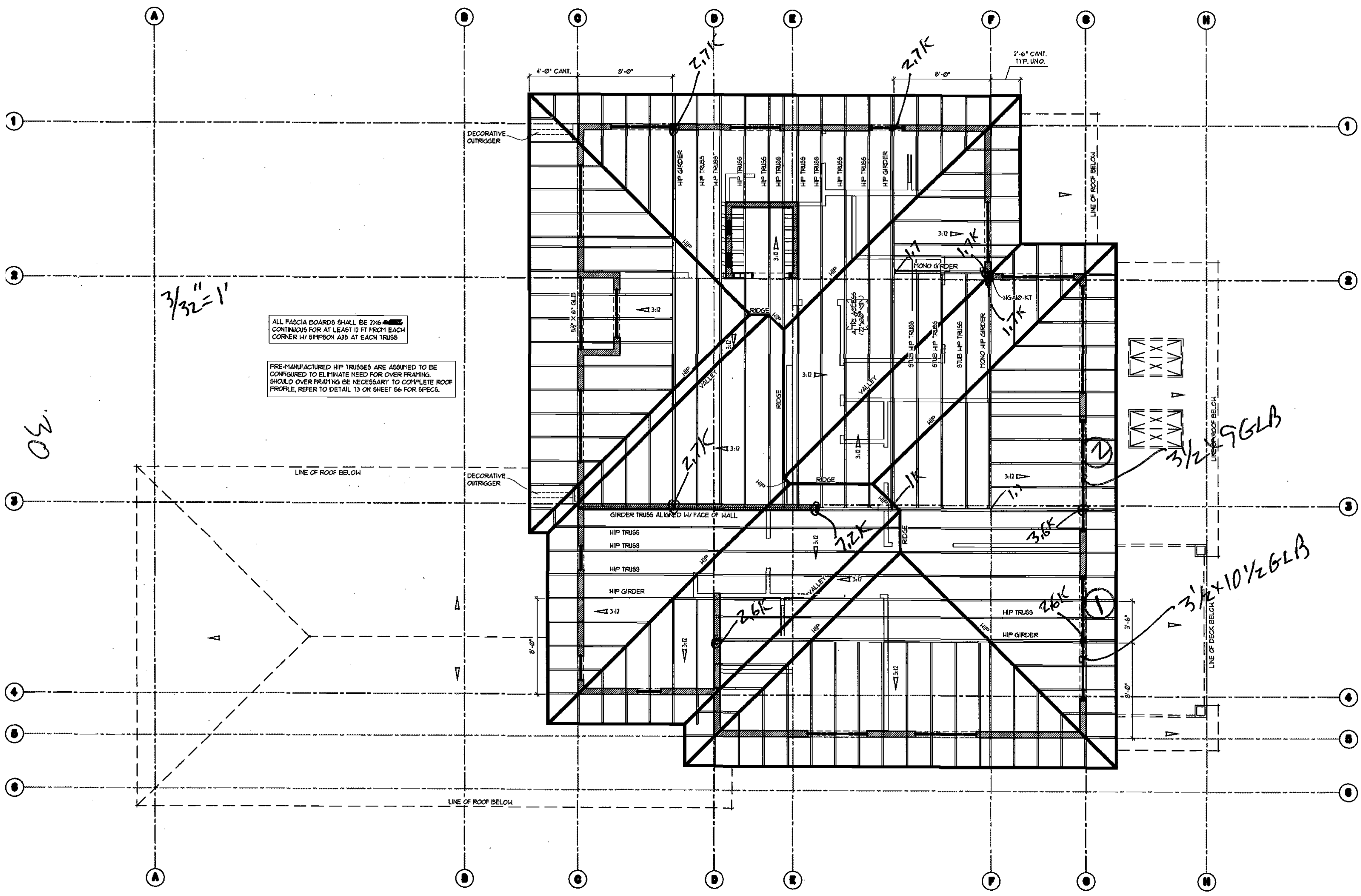
$\frac{vg \cdot Lg}{42.5ft} = 63.69\,lb \cdot ft^{-1}$ $\quad \frac{E_g \cdot Lg}{42.5ft} = 57.26\,lb \cdot ft^{-1}$

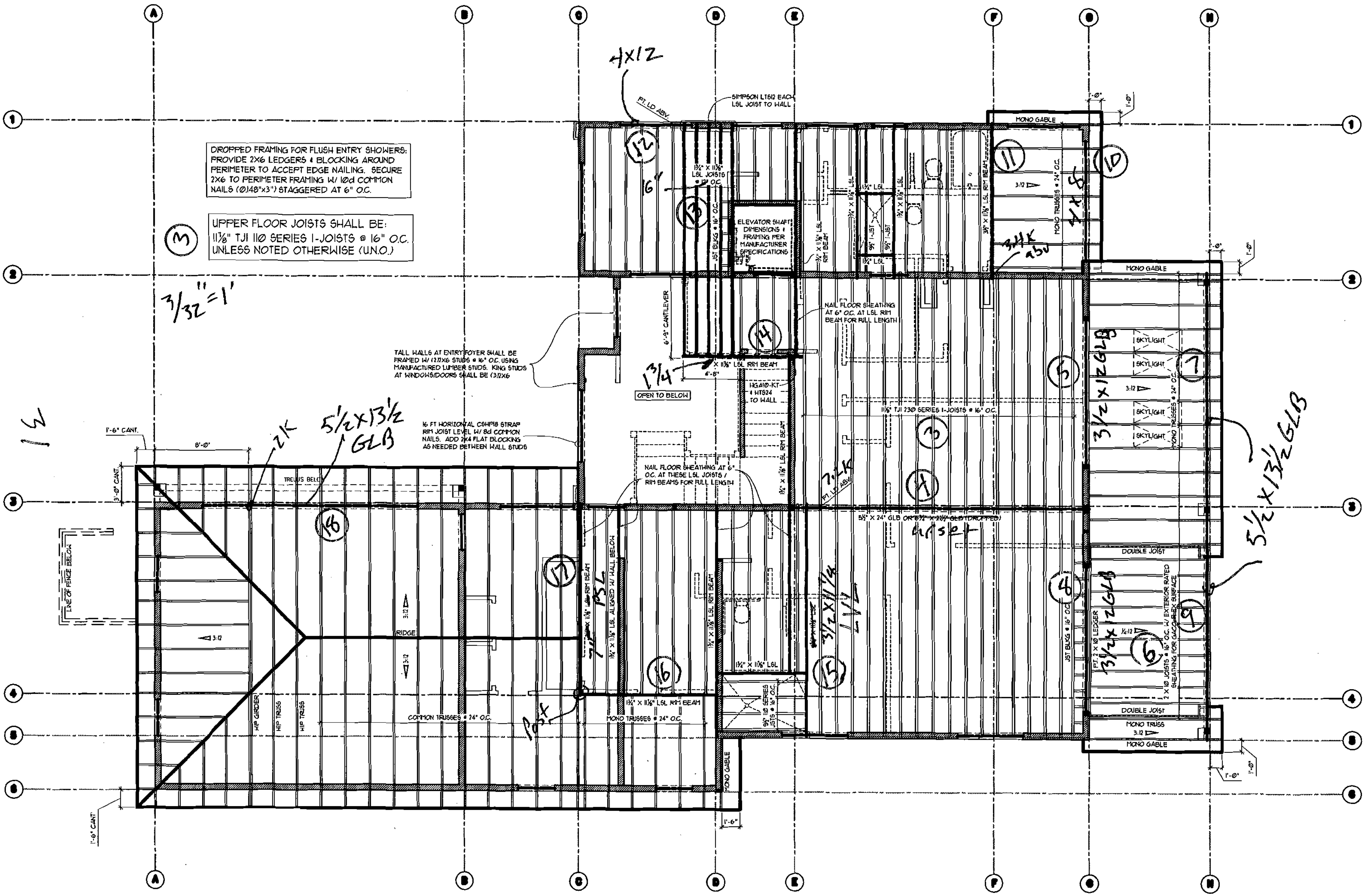
Wall Line H:

$\frac{vh \cdot Lh}{24ft} = 120.11\,lb \cdot ft^{-1}$ $\quad \frac{E_h \cdot Lh}{24ft} = 80.76\,lb \cdot ft^{-1}$

Wall Line I:

$\frac{vi \cdot Li}{24ft} = 87.97\,lb \cdot ft^{-1}$ $\quad \frac{E_i \cdot Li}{24ft} = 59.15\,lb \cdot ft^{-1}$


ALL FASCIA BOARDS SHALL BE 2X6 CONTINUOUS FOR AT LEAST 12 FT FROM EACH CORNER W/ SIMPSON A35 AT EACH TRUSS
PRE-MANUFACTURED HIP TRUSSES ARE ASSUMED TO BE CONFIGURED TO ELIMINATE NEED FOR OVER FRAMING. SHOULD OVER FRAMING BE NECESSARY TO COMPLETE ROOF PROFILE, REFER TO DETAIL 13 ON SHEET S6 FOR SPECS.
3/32" = 1'
30
4'-0" CANT.
8'-0"
2'-6" CANT. TYP. U.N.O.
DECORATIVE OUTRIGGER
LINE OF ROOF BELOW
LINE OF DECK BELOW
HIP GIRDER
HIP TRUSS
STUB HIP TRUSS
MONO HIP GIRDER
MONO GIRDER
GIRDER TRUSS ALIGNED W/ FACE OF WALL
HIP
VALLEY
RIDGE
3:12
HGA10-KT
5¼" X 6" GLB
ATTIC ACCESS (22"x30" MIN)
3'-6"
2.7K
1.7K
1.1
1K
7.2K
3.6K
2.6K
3½x9GLB
3½x10½GLB


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 (Ø.148"x3") STAGGERED AT 6" O.C.
3
UPPER FLOOR JOISTS SHALL BE: 11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C. UNLESS NOTED OTHERWISE (U.N.O.)
3/32" = 1'
4X12
PT. LD ABV
SIMPSON LTS12 EACH LSL JOIST TO WALL
MONO GABLE
1'-0"
1¾" X 11⅞" LSL JOISTS @ 16" O.C.
6"
JST BLKG @ 16" O.C.
ELEVATOR SHAFT DIMENSIONS & FRAMING PER MANUFACTURER SPECIFICATIONS
1¾" X 11⅞" LSL RIM BEAM
9½" I-JST
1¾" LSL
3½" X 11⅞" LSL RIM BEAM
MONO TRUSSES @ 24" O.C.
3:12
4X8
3-4K abv
6'-9" CANTILEVER
NAIL FLOOR SHEATHING AT 6" O.C. AT LSL RIM BEAM FOR FULL LENGTH
TALL WALLS AT ENTRY FOYER SHALL BE FRAMED W/ (2)2X6 STUDS @ 16" O.C. USING MANUFACTURED LUMBER STUDS. KING STUDS AT WINDOWS/DOORS SHALL BE (3)2X6
1¾" X 11⅞" LSL RIM BEAM
1¾
4'-8"
HGA10-KT & H2.5A TO WALL
OPEN TO BELOW
SKYLIGHT
3½X12 GLB
11⅞" TJI 230 SERIES I-JOISTS @ 16" O.C.
16 FT HORIZONTAL CS16 STRAP RIM JOIST LEVEL W/ 8d COMMON NAILS. ADD 2X4 FLAT BLOCKING AS NEEDED BETWEEN WALL STUDS
1'-6" CANT.
8'-0"
2K
5½X13½ GLB
3'-0" CANT.
TRELLIS BELOW
NAIL FLOOR SHEATHING AT 6" O.C. AT THESE LSL JOISTS / RIM BEAMS FOR FULL LENGTH
7-K
PT. LD ABV
5½" X 24" GLB OR 8¾" X 22½" GLB (DROPPED)
4-1-5 PT
5½X13½GLB
DOUBLE JOIST
LINE OF FENCE BELOW
PSL
1¾" X 11⅞" LSL ALIGNED W/ WALL BELOW
1¾" X 11⅞" LSL RIM BEAM
3½X11¼ LVL
JST BLKG @ 16" O.C.
PT. 2 X 10 LEDGER
¼:12
3½X12GLB
2 X 10 JOISTS @ 16" O.C. W/ EXTERIOR RATED SHEATHING FOR GACO-FLEX SURFACE
3:12
RIDGE
1¾" X 11⅞" LSL
9½" 110 SERIES JSTS @ 16" O.C.
HIP GIRDER
HIP TRUSS
COMMON TRUSSES @ 24" O.C.
Post
MONO TRUSSES @ 24" O.C.
MONO TRUSS
MONO GABLE
1'-6"
1'-6" CANT.
A
B
C
D
E
F
G
H
1
2
3
4
5
6
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18

31

3/32" = 1'

CRAWL SPACE # 1 VENTILATION

$\frac{\text{CRAWL AREA}}{300}$ = NET VENT AREA REQ'D (N.V.A.) (ASSUMES CROSS VENTILATION)

$\frac{2367}{300}$ = 7.89 SQ. FT. N.V.A. REQUIRED

IF 14" x 7" SCREENED FOUNDATION VENTS USED

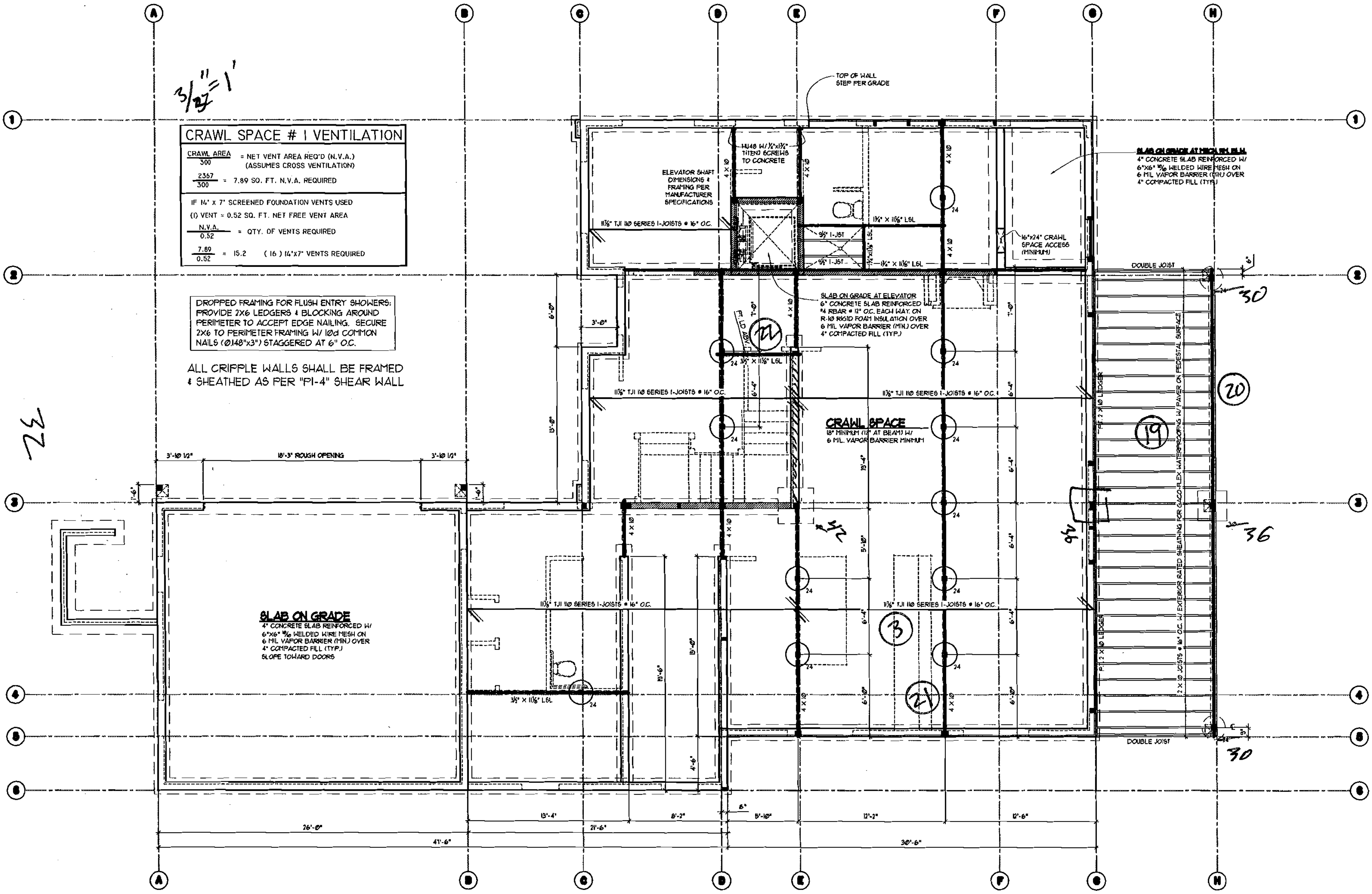
(1) VENT = 0.52 SQ. FT. NET FREE VENT AREA

$\frac{\text{N.V.A.}}{0.52}$ = QTY. OF VENTS REQUIRED

$\frac{7.89}{0.52}$ = 15.2 (16) 14"x7" VENTS REQUIRED

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" O.C.

ALL CRIPPLE WALLS SHALL BE FRAMED & SHEATHED AS PER "PI-4" SHEAR WALL



32

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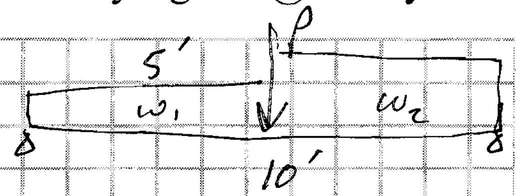
myengineer@centurytel.net

① $w_{D_1} = 15psf(13'/2) = 97.5plf$
$w_{S_1} = 25psf(13'/2) = 162.5plf$

$w_{D_2} = 15psf(36'/2) = 270plf$
$w_{S_2} = 25psf(36'/2) = 450plf$

$P = 1052^{\#}DL + 1548^{\#}SL$ from girder

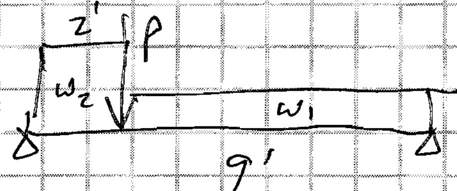


3½x10½ GLB

② $w_{D_1} = 97.5plf$
$w_{S_1} = 162.5plf$

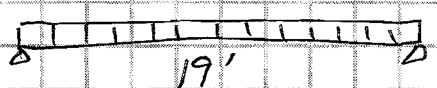
$w_{D_2} = 15psf(47.5'/2) = 356.3plf$
$w_{S_2} = 25psf(47.5'/2) = 593.8plf$

$P = 1440^{\#}DL + 2160^{\#}SL$ from girder



3½x9 GLB

③ $w_D = 15psf$
$w_L = 40psf$

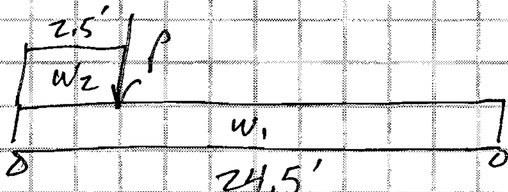


11⅞ TJI 210 @ 16" o.c.
(TJI 110 @ 17'8" span)

④ $w_{D_1} = 15psf(38'/2) = 285plf$
$w_{L_1} = 40psf(38'/2) = 760plf$

$w_{D_2} = 15psf(34'/2) = 255plf$
$w_{S_2} = 25psf(34'/2) = 425plf$

$P = 2900^{\#}DL + 4300^{\#}SL$ from Girder



5½x24 GLB
OR
8¾x21 GLB
OR
W12x50 A992 Steel
(8.08"w x 12.2"D)

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY MM

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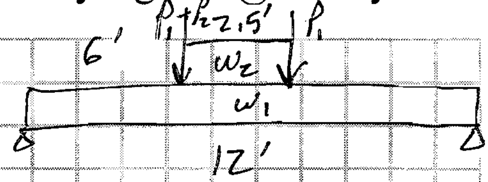
myengineer@centurytel.net

(5) $w_{D_1} = 15psf(1' + 10'/2) + 20psf(9') = 270plf$
$w_{L_1} = 40plf$
$w_{S_1} = 25psf(10'/2) = 125plf$

$w_{D2} = 15psf(13'/2) = 97.5plf$
$w_{S2} = 25psf(13'/2) = 162.5plf$

$P_1 = 440^{\#}DL + 730^{\#}SL$ from header

$P_2 = \pm 600^{\#}WL \pm 1220^{\#}EL$

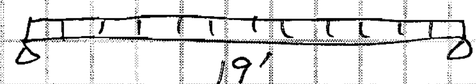


3½x12 GLB
OR
3½x11⅞ LSL

(6) $w_D = 15psf$
$w_L = 60psf$
$w_S = 25psf$

P.T. 2x10 @ 16" o.c.

(7) $w_D = 15psf(12'/2) = 90plf$
$w_S = 25psf(12'/2) = 150plf$

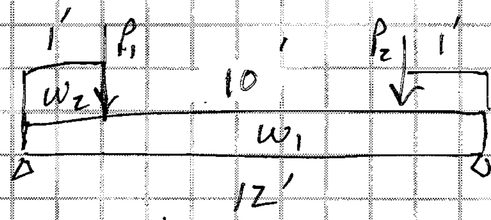


5½x12 GLB
Minimum

(8) $w_{D_1} = 15psf(1' + 10'/2) + 20psf(9') = 270plf$
$w_{L_1} = 40plf + 60psf(10'/2) = 340plf$
$w_{S_1} = 25psf(10'/2) = 125plf$

$w_{D2} = 15psf(47.5'/2) = 356.3plf$
$w_{S2} = 25psf(47.5'/2) = 593.8plf$

$P_1 = 1230^{\#}DL + 1950^{\#}SL$ from (1)
$\pm 600^{\#}WL \pm 1220^{\#}EL$
$P_2 = 1660^{\#}DL + 2660^{\#}SL$ from (1)
$\pm 1375^{\#}WL \pm 2865^{\#}EL$



3½x12 GLB
OR
3½x11⅞ LSL

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY MM

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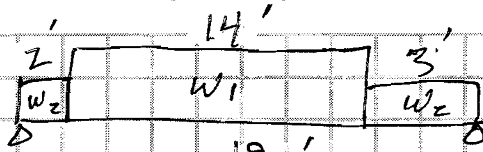
myengineer@centurytel.net

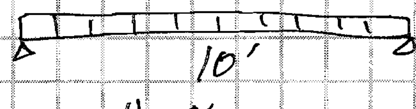
(9) $w_{D1} = 15psf(10'/2) = 75plf$
$w_{L1} = 60psf(10'/2) = 300plf$
$w_{S1} = 25psf(10'/2) = 125plf$

$w_{D2} = 15psf(12'/2) = 90plf$
$w_{S2} = 25psf(12'/2) = 150plf$



5½x13½ GLB

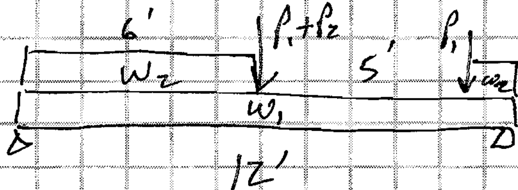
(10) $w_D = 15psf(10'/2) = 75plf$
$w_S = 25psf(10'/2) = 125plf$



4x8

(11) $w_{D1} = 15psf(1' + 8'/2) + 20psf(9') = 255plf$
$w_{L1} = 40plf$
$w_{S1} = 25psf(8'/2) = 100plf$

$w_{D2} = 15psf(13'/2) = 97.5plf$
$w_{S2} = 25psf(13'/2) = 162.5plf$



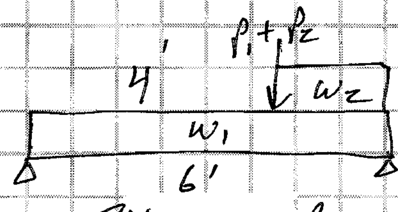
3½x11⅞ LSL

$P_1 = 245^{\#}DL + 405^{\#}SL$ from header

$P_2 = \pm 1375^{\#}WL \pm 2865^{\#}EL$ from shearwall $\Omega = 3.0$

(12) $w_{D1} = 15psf(13'/2) + 20psf(9') = 277.5plf$
$w_{L1} = 40psf(13'/2) = 260plf$

$w_{D2} = 15psf(37'/2) = 277.5plf$
$w_{S2} = 25psf(37'/2) = 462.5plf$



3½x9 GLB

$P_1 = 245^{\#}DL + 405^{\#}SL$ from header
$\pm 1110^{\#}WL \pm 2060^{\#}EL$
$P_2 = 1080^{\#}DL + 1620^{\#}SL$ from girder

(4x10 w/ full uniform & No Pt. Ld.)

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY MM

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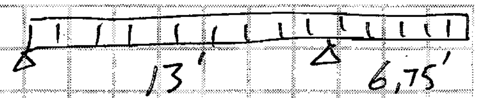
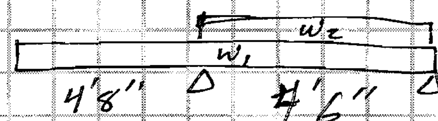
(13) $w_D = 15\,psf$
$w_L = 40\,psf$



1 3/4 x 11 7/8 LSL @ 16" o.c.

(14) $w_{D1} = 15\,psf(7'/2) = 52.5\,plf$
$w_{L1} = 40\,psf(7'/2) = 140\,plf$

$w_{D2} = 15\,psf(8'/2) = 60\,plf$
$w_{L2} = 40\,psf(8'/2) = 160\,plf$

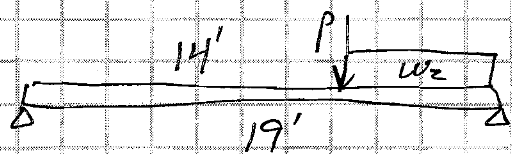


1 3/4 x 11 7/8 LSL

(15) $w_{D1} = 15\,plf$
$w_{L1} = 40\,plf$

$w_{D2} = 15\,plf(7'/2) = 52.5\,plf$
$w_{L2} = 40\,plf(7'/2) = 140\,plf$

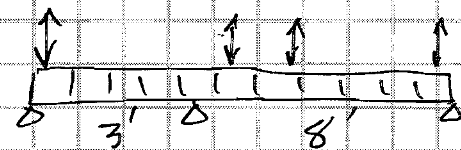
$P = 370^{\#} DL + 980^{\#} LL$



3 1/2 x 11 7/8 LVL

(16) $w_D = 15\,psf(16'/2 + 8'/2 + 13'/2) + 20\,psf(9') = 457.5\,plf$
$w_L = 40\,psf(16'/2) = 320\,plf$
$w_S = 25\,psf(8'/2 + 13'/2) = 262.5\,plf$

$P = \pm 805^{\#} WL \pm 1490^{\#} EL$ w/ $\Omega = 3.0$



1 3/4 x 11 7/8 LSL

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY MM

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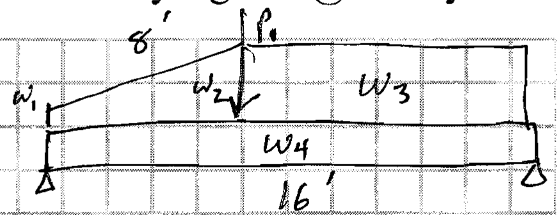


7x11 7/8 PSL

(17) $w_{D1} = 15psf(2.5') = 37.5plf$
$w_{S1} = 25psf(2.5') = 62.5plf$

$w_{D2} = 15psf(13'/2) = 97.5plf$
$w_{S2} = 25psf(13'/2) = 162.5plf$

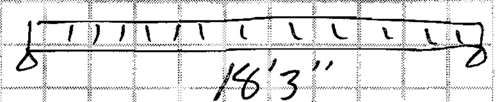
$P_1 = 400^{\#}DL + 600^{\#}SL$ from girder

$w_{D3} = 15psf(47.5'/2) = 356.3plf$
$w_{S3} = 25psf(47.5'/2) = 593.8plf$

$P_2 = \pm 1000^{\#} WL \pm 2000^{\#} EL$

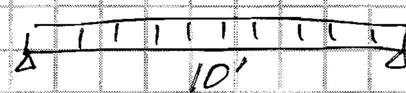
$w_{D4} = 15plf(2') + 20psf(9') = 210plf$
$w_{L4} = 40plf$
$w_{S4} = 25plf$

(18) $w_D = 15psf(30'/2) = 225plf$
$w_S = 25psf(30'/2) = 375plf$



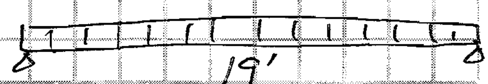
5 1/2 x 13 1/2 GLB

(19) $w_D = 25psf$
$w_L = 60psf$



P.T. 2x10 @ 16" o.c.

(20) $w_D = 25psf(10'/2) = 125plf$
$w_L = 60psf(10'/2) = 300plf$



5 1/2 x 13 1/2 GLB

(P.T. 6x12 @ 10' span)

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY [signature]

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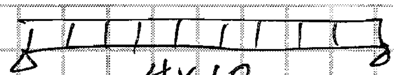
(21) $w_D = 15psf\left(\frac{25'}{2}\right) = 187.5plf$
$w_L = 40psf\left(\frac{25'}{2}\right) = 500plf$

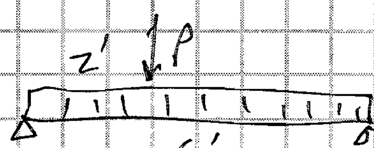


4x10

7'2" clear span
Max

(22) $w_D = 15psf$
$w_L = 40psf$

$P = 630^{\#}DL + 1670^{\#}LL$ from (14)



6'

$1\frac{3}{4} \times 11\frac{7}{8}$ LSL
Minimum

FOR RFA/RKK
JOB Masin

DATE 10-20-21
BY MM

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Mark Myers, PE
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Gig Harbor, WA 98335

Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 1. Header at Master/Deck

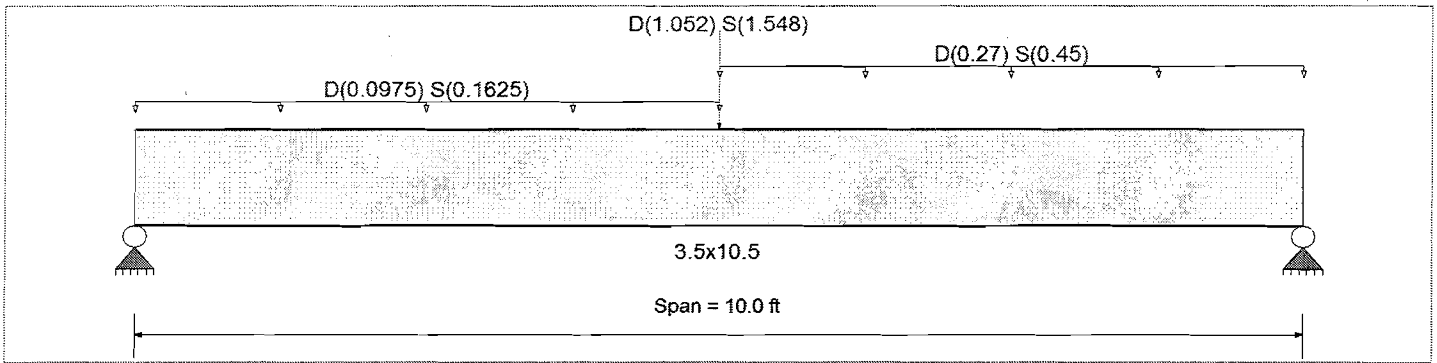
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 0.0 -->> 5.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.270, S = 0.450 k/ft, Extent = 5.0 -->> 10.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.052, S = 1.548 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.854** : 1	Maximum Shear Stress Ratio	=	**0.498** : 1
Section used for this span		**3.5x10.5**	Section used for this span		**3.5x10.5**
	=	2,355.69 psi		=	151.86 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	9.161 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.206 in	Ratio =	581 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.337 in	Ratio =	355 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.175	4.325
Overall MINimum	1.946	2.665
D Only	1.229	1.660
+D+L	1.229	1.660
+D+S	3.175	4.325
+D+0.750L	1.229	1.660
+D+0.750L+0.750S	2.689	3.659
+0.60D	0.737	0.996
S Only	1.946	2.665

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Gig Harbor, WA 98335

DESCRIPTION: 2. Header at Den

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

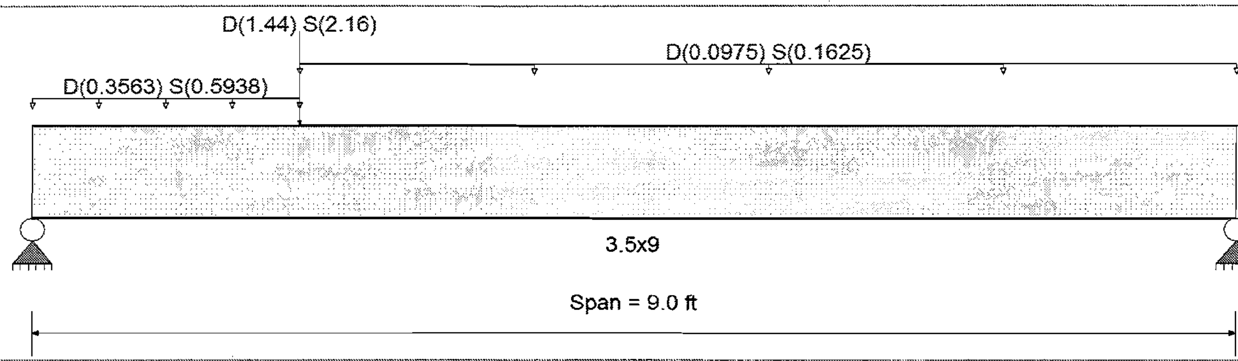
Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Ebend- yy	1,600.0 ksi
Eminbend - yy	850.0 ksi
Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.3563, S = 0.5938 k/ft, Extent = 0.0 -->> 2.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 2.0 -->> 9.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.440, S = 2.160 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.781: 1	Maximum Shear Stress Ratio	=	0.705 : 1
Section used for this span		**3.5x9**	Section used for this span		**3.5x9**
	=	2,156.79 psi		=	214.78 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.004 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.176 in	Ratio =	611 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.289 in	Ratio =	374 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.197	2.123
Overall MINimum	3.178	1.307
D Only	2.019	0.816
+D+L	2.019	0.816
+D+S	5.197	2.123
+D+0.750L	2.019	0.816
+D+0.750L+0.750S	4.402	1.797
+0.60D	1.211	0.490
S Only	3.178	1.307

3

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.
9½"	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"
11⅞"	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)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

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.
9½"	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"
11⅞"	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)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"(1)***	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"(1)***	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"(1)***	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"(1)***	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"(1)***	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 5¼" 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"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			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".

How to Use These Tables

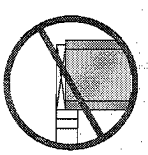
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

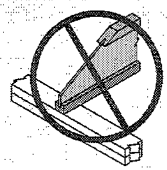
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor 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.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

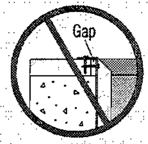
These Conditions Are NOT Permitted:



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.

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Wood Beam

DESCRIPTION: 4. Floor Beam at Grid 3 (Wood)

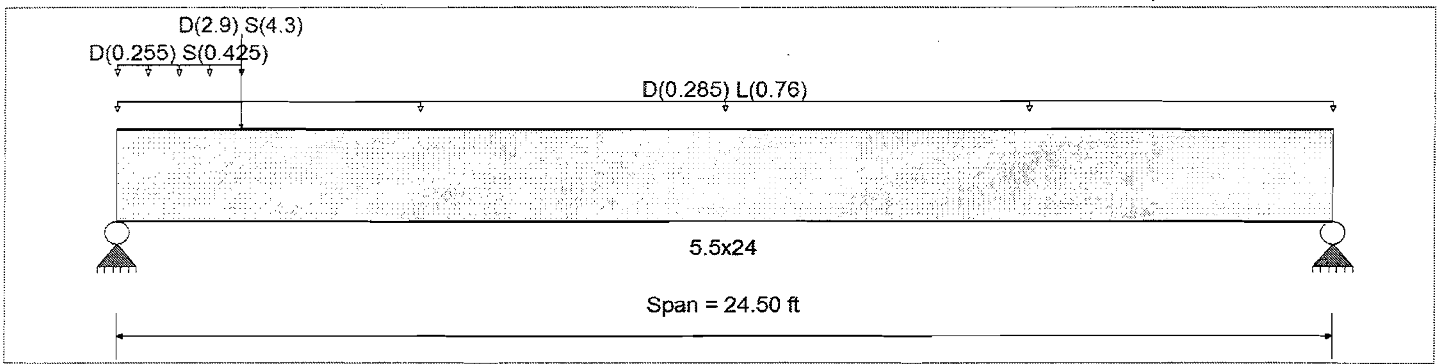
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 19.0 ft
Uniform Load : D = 0.2550, S = 0.4250 k/ft, Extent = 0.0 -->> 2.50 ft, Tributary Width = 1.0 ft
Point Load : D = 2.90, S = 4.30 k @ 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.856 : 1**	Maximum Shear Stress Ratio	=	**0.577 : 1**
Section used for this span		**5.5x24**	Section used for this span		**5.5x24**
	=	1,874.58 psi		=	152.88 psi
	=	2,189.51 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	11.892 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.543 in	Ratio =	541 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.793 in	Ratio =	370 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	17.335	13.130
Overall MINimum	4.870	0.493
D Only	6.700	3.820
+D+L	16.010	13.130
+D+S	11.570	4.313
+D+0.750L	13.683	10.802
+D+0.750L+0.750S	17.335	11.172
+0.60D	4.020	2.292
L Only	9.310	9.310
S Only	4.870	0.493

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Wood Beam

DESCRIPTION: 4. Floor Beam at Grid 3 (Wood)

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

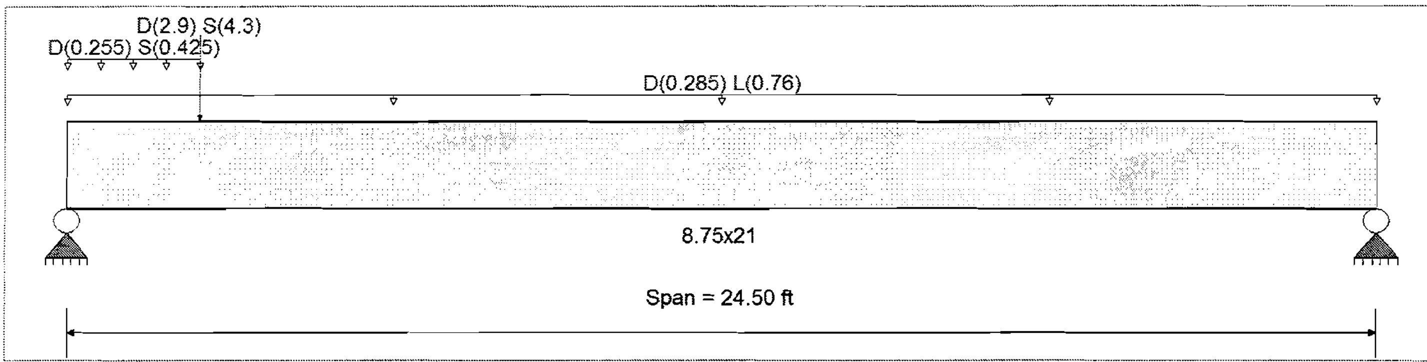
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.210 pcf



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 19.0 ft
Uniform Load : D = 0.2550, S = 0.4250 k/ft, Extent = 0.0 -->> 2.50 ft, Tributary Width = 1.0 ft
Point Load : D = 2.90, S = 4.30 k @ 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.727: 1	Maximum Shear Stress Ratio	=	0.425 : 1
Section used for this span		**8.75x21**	Section used for this span		**8.75x21**
	=	1,539.02 psi		=	112.67 psi
	=	2,118.27 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	11.892 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.510 in	Ratio =	576 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.744 in	Ratio =	395 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	17.335	13.130
Overall MINimum	4.870	0.493
D Only	6.700	3.820
+D+L	16.010	13.130
+D+S	11.570	4.313
+D+0.750L	13.683	10.802
+D+0.750L+0.750S	17.335	11.172
+0.60D	4.020	2.292
L Only	9.310	9.310
S Only	4.870	0.493

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Steel Beam

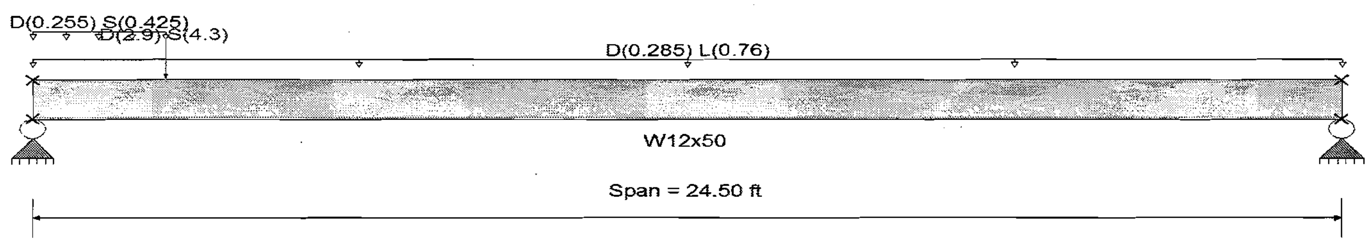
DESCRIPTION: 4a. Floor Beam at Grid 3 (Steel)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 19.0 ft

Uniform Load : D = 0.2550, S = 0.4250 k/ft, Extent = 0.0 -->> 2.50 ft, Tributary Width = 1.0 ft

Point Load : D = 2.90, S = 4.30 k @ 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.674** : 1	Maximum Shear Stress Ratio =	**0.200** : 1
Section used for this span	**W12x50**	Section used for this span	**W12x50**
Ma : Applied	82.482 k-ft	Va : Applied	18.038 k
Mn / Omega : Allowable	122.373 k-ft	Vn/Omega : Allowable	90.280 k
Load Combination	+D+L	Load Combination	+1.105D+0.750L+0.750S
Location of maximum on span	11.970ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection			
Max Downward Transient Deflection	0.546 in	Ratio =	538 >=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480.0
Max Downward Total Deflection	0.796 in	Ratio =	369 >=360.
Max Upward Total Deflection	0.000 in	Ratio =	0 <360.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.7963	12.180		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	17.335	13.130
Overall MINimum	4.020	0.493
D Only	6.700	3.820
+D+L	16.010	13.130
+D+S	11.570	4.313
+D+0.750L	13.683	10.802
+D+0.750L+0.750S	17.335	11.172
+0.60D	4.020	2.292
L Only	9.310	9.310
S Only	4.870	0.493

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Steel Beam

DESCRIPTION: 4a. Floor Beam at Grid 3 (Steel)

Steel Section Properties : W12x50

Depth	=	12.200	in	I xx	=	391.00	in^4	J	=	1.710	in^4
Web Thick	=	0.370	in	S xx		64.20	in^3	Cw	=	1,880.00	in^6
Flange Width	=	8.080	in	R xx	=	5.180	in				
Flange Thick	=	0.640	in	Zx	=	71.900	in^3				
Area	=	14.600	in^2	I yy	=	56.300	in^4				
Weight	=	50.000	plf	S yy	=	13.900	in^3	Wno	=	23.400	in^2
Kdesign	=	1.140	in	R yy	=	1.960	in	Sw	=	30.200	in^4
K1	=	0.938	in	Zy	=	21.300	in^3	Qf	=	14.300	in^3
rts	=	2.250	in					Qw	=	35.400	in^3
Ycg	=	6.100	in								

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DESCRIPTION: 5. Header at Great Rm

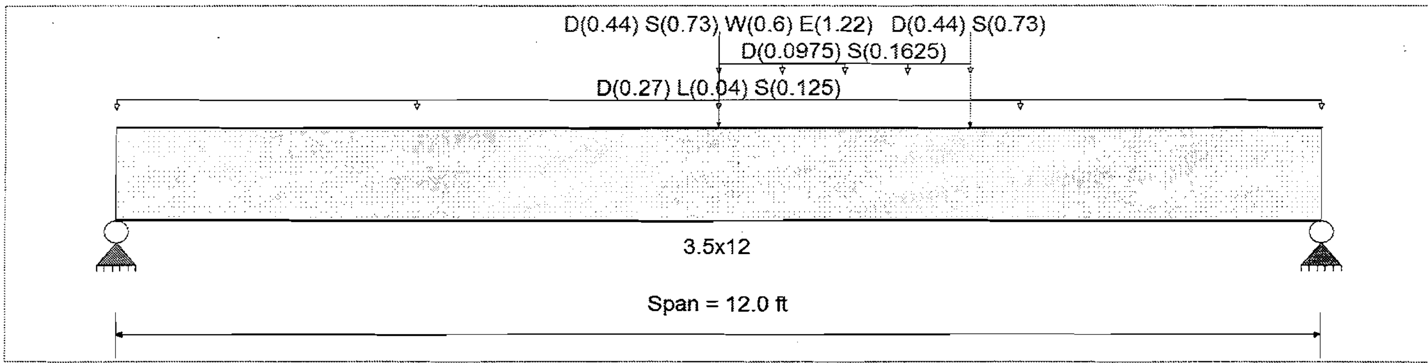
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Uniform Load : D = 0.270, L = 0.040, S = 0.1250 , Tributary Width = 1.0 ft
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 6.0 -->> 8.50 ft, Tributary Width = 1.0 ft
Point Load : D = 0.440, S = 0.730, W = 0.60, E = 1.220 k @ 6.0 ft
Point Load : D = 0.440, S = 0.730 k @ 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.736** : 1	Maximum Shear Stress Ratio	=	**0.445** : 1
Section used for this span		**3.5x12**	Section used for this span		**3.5x12**
	=	2,030.18 psi		=	135.57 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.000 ft	Location of maximum on span	=	11.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.180 in	Ratio =	799 >=360
Max Upward Transient Deflection	-0.084 in	Ratio =	1711 >=360
Max Downward Total Deflection	0.404 in	Ratio =	356 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.682	4.207
Overall MINimum	-0.610	-0.610
D Only	2.065	2.299
+D+L	2.305	2.539
+D+S	3.554	4.176
+D+0.750L	2.245	2.479
+D+0.750L+0.750S	3.361	3.887
+D+0.60W	2.245	2.479
+D-0.60W	1.885	2.119
+D+0.70E	2.492	2.726
+D-0.70E	1.638	1.872
+D+0.750L+0.450W	2.380	2.614

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Wood Beam

DESCRIPTION: 5. Header at Great Rm

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L-0.450W	2.110	2.344
+D+0.750L+0.750S+0.450W	3.496	4.022
+D+0.750L+0.750S-0.450W	3.226	3.752
+D+0.750L+0.750S+0.5250E	3.682	4.207
+D+0.750L+0.750S-0.5250E	3.041	3.567
+0.60D+0.60W	1.419	1.559
+0.60D-0.60W	1.059	1.199
+0.60D+0.70E	1.666	1.806
+0.60D-0.70E	0.812	0.952
L Only	0.240	0.240
S Only	1.489	1.878
W Only	0.300	0.300
-W	-0.300	-0.300
E Only	0.610	0.610
E Only * -1.0	-0.610	-0.610

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Wood Beam

File: Masin Residence.ec6

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DESCRIPTION: 5a. Rim Beam at Great Rm

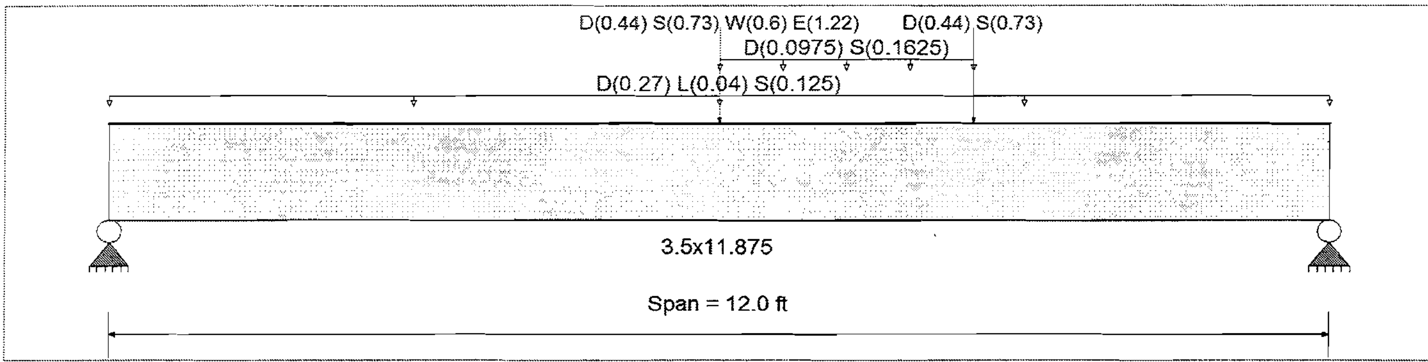
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,050.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf

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



Applied Loads

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

Uniform Load : D = 0.270, L = 0.040, S = 0.1250 , Tributary Width = 1.0 ft
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 6.0 -->> 8.50 ft, Tributary Width = 1.0 ft
Point Load : D = 0.440, S = 0.730, W = 0.60, E = 1.220 k @ 6.0 ft
Point Load : D = 0.440, S = 0.730 k @ 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.775** 1	Maximum Shear Stress Ratio	=	**0.384** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	2,073.14 psi		=	136.99 psi
	=	2,673.75 psi		=	356.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.000 ft	Location of maximum on span	=	11.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.216 in	Ratio =	667 >=360
Max Upward Transient Deflection	-0.101 in	Ratio =	1428 >=360
Max Downward Total Deflection	0.484 in	Ratio =	297 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.682	4.207
Overall MINimum	-0.610	-0.610
D Only	2.065	2.299
+D+L	2.305	2.539
+D+S	3.554	4.176
+D+0.750L	2.245	2.479
+D+0.750L+0.750S	3.361	3.887
+D+0.60W	2.245	2.479
+D-0.60W	1.885	2.119
+D+0.70E	2.492	2.726
+D-0.70E	1.638	1.872
+D+0.750L+0.450W	2.380	2.614

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DESCRIPTION: 5a. Rim Beam at Great Rm

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L-0.450W	2.110	2.344
+D+0.750L+0.750S+0.450W	3.496	4.022
+D+0.750L+0.750S-0.450W	3.226	3.752
+D+0.750L+0.750S+0.5250E	3.682	4.207
+D+0.750L+0.750S-0.5250E	3.041	3.567
+0.60D+0.60W	1.419	1.559
+0.60D-0.60W	1.059	1.199
+0.60D+0.70E	1.666	1.806
+0.60D-0.70E	0.812	0.952
L Only	0.240	0.240
S Only	1.489	1.878
W Only	0.300	0.300
-W	-0.300	-0.300
E Only	0.610	0.610
E Only * -1.0	-0.610	-0.610

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Wood Beam

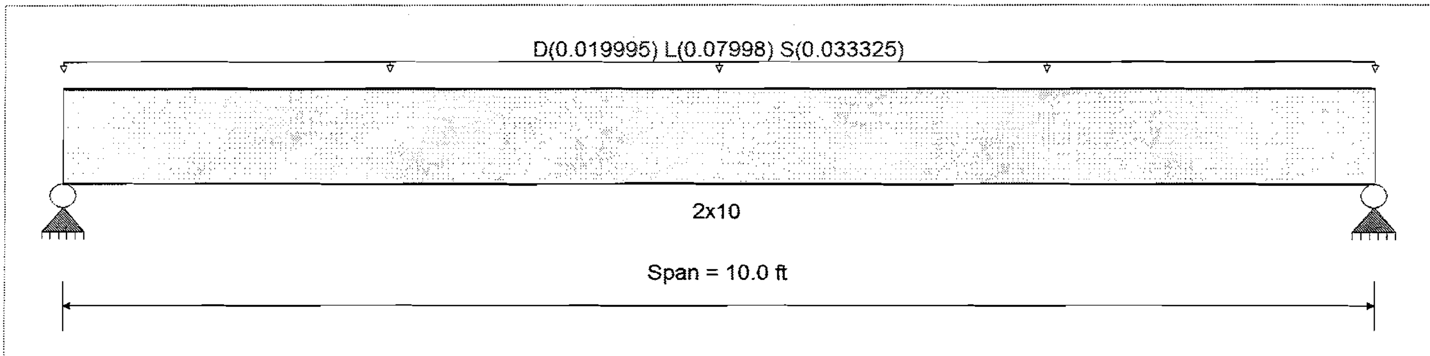
DESCRIPTION: 6. Upper Deck Joists

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850 psi	Ebend- xx	1300 ksi
		Fc - Prll	1300 psi	Eminbend - xx	470 ksi
Wood Species	: Hem-Fir	Fc - Perp	405 psi		
Wood Grade	: No.2	Fv	150 psi		
		Ft	525 psi	Density	26.84 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.060, S = 0.0250 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.815: 1	Maximum Shear Stress Ratio	=	**0.381 : 1**
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	701.07 psi		=	45.76 psi
	=	860.20 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.148 in	Ratio =	810 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.194 in	Ratio =	617 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.525	0.525
Overall MINimum	0.167	0.167
D Only	0.100	0.100
+D+L	0.500	0.500
+D+S	0.267	0.267
+D+0.750L	0.400	0.400
+D+0.750L+0.750S	0.525	0.525
+0.60D	0.060	0.060
L Only	0.400	0.400
S Only	0.167	0.167

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Wood Beam

DESCRIPTION: 7. COvered Porch Roof Beam at Great Rm

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

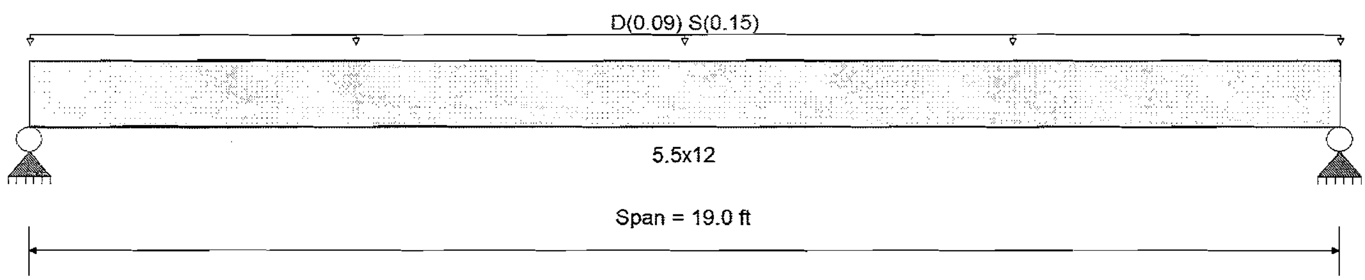
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2400 psi	*E : Modulus of Elasticity*	
Fb -	1850 psi	Ebend- xx	1800 ksi
Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Fc - Perp	650 psi	Ebend- yy	1600 ksi
Fv	265 psi	Eminbend - yy	850 ksi
Ft	1100 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.090, S = 0.150 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.357**: 1	Maximum Shear Stress Ratio	=	**0.153** : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	984.55 psi		=	46.52 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.500 ft	Location of maximum on span	=	18.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.310 in	Ratio =	734 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.497 in	Ratio =	459 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.280	2.280
Overall MINimum	1.425	1.425
D Only	0.855	0.855
+D+L	0.855	0.855
+D+S	2.280	2.280
+D+0.750L	0.855	0.855
+D+0.750L+0.750S	1.924	1.924
+0.60D	0.513	0.513
S Only	1.425	1.425

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Wood Beam

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DESCRIPTION: 8. Header at Dining Rm

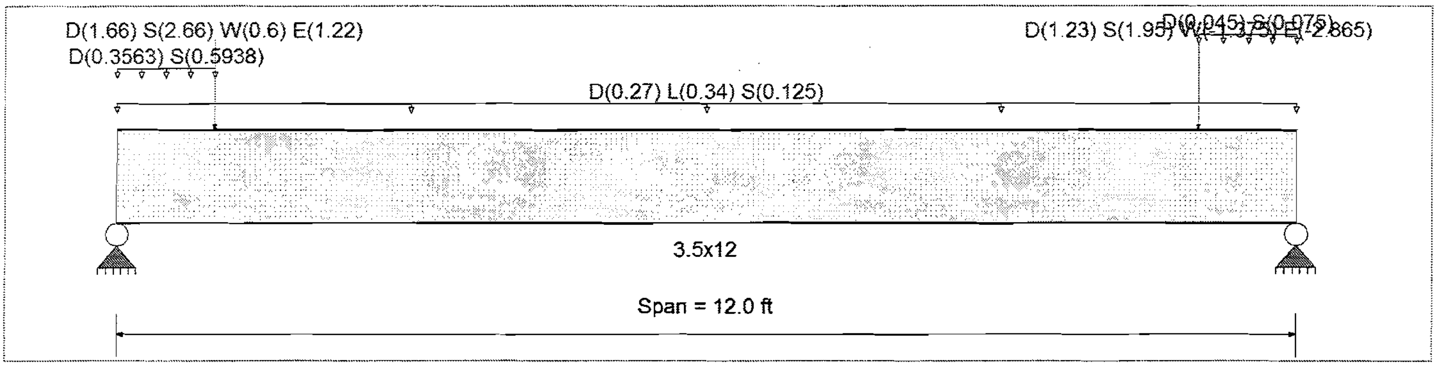
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Uniform Load : D = 0.270, L = 0.340, S = 0.1250 , Tributary Width = 1.0 ft
Uniform Load : D = 0.3563, S = 0.5938 k/ft, Extent = 0.0 -->> 1.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.230, S = 1.950, W = -1.375, E = -2.865 k @ 11.0 ft
Point Load : D = 1.660, S = 2.660, W = 0.60, E = 1.220 k @ 1.0 ft
Uniform Load : D = 0.0450, S = 0.0750 k/ft, Extent = 11.0 -->> 12.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.753** 1	Maximum Shear Stress Ratio	=	**0.860** : 1
Section used for this span		**3.5x12**	Section used for this span		**3.5x12**
	=	2,078.09 psi		=	364.45 psi
	=	2,760.00 psi		=	424.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+1.116D+0.750L+0.750S-1.575E
Location of maximum on span	=	5.825 ft	Location of maximum on span	=	11.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.176 in	Ratio =	818 >=360
Max Upward Transient Deflection	-0.032 in	Ratio =	4551 >=360
Max Downward Total Deflection	0.451 in	Ratio =	319 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.522	7.941
Overall MINimum	-0.880	2.525
D Only	3.587	2.944
+D+L	5.627	4.984
+D+S	7.511	5.800
+D+0.750L	5.117	4.474
+D+0.750L+0.750S	8.060	6.616
+D+0.60W	3.849	2.218
+D-0.60W	3.326	3.670
+D+0.70E	4.203	1.177
+D-0.70E	2.972	4.711

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DESCRIPTION: 8. Header at Dining Rm

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.450W	5.313	3.929
+D+0.750L-0.450W	4.922	5.018
+D+0.750L+0.750S+0.450W	8.256	6.071
+D+0.750L+0.750S-0.450W	7.864	7.160
+D+0.750L+0.750S+0.5250E	8.522	5.290
+D+0.750L+0.750S-0.5250E	7.598	7.941
+0.60D+0.60W	2.414	1.040
+0.60D-0.60W	1.891	2.493
+0.60D+0.70E	2.768	-0.001
+0.60D-0.70E	1.537	3.533
L Only	2.040	2.040
S Only	3.923	2.856
W Only	0.435	-1.210
-W	-0.435	1.210
E Only	0.880	-2.525
E Only * -1.0	-0.880	2.525

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Wood Beam

DESCRIPTION: 8a. Rim Beam at Dining Rm

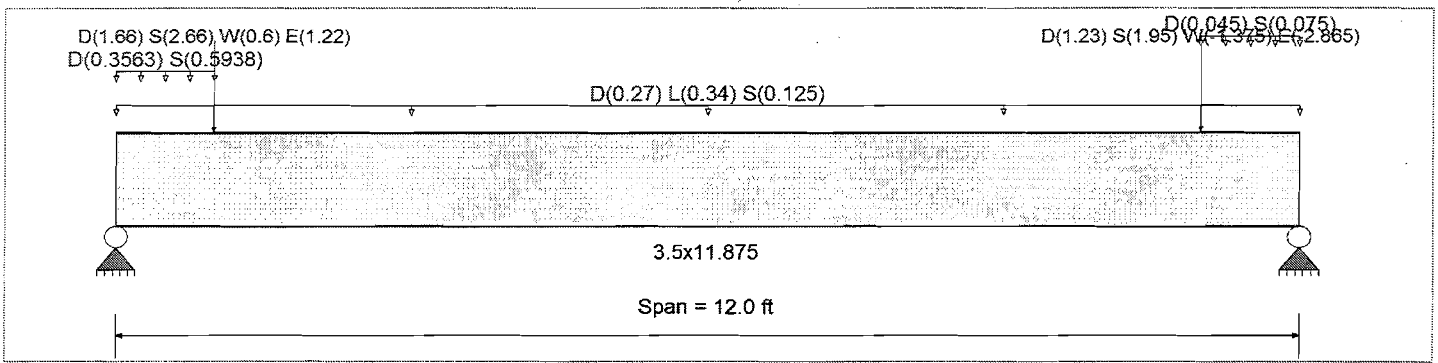
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,050.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf

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



Applied Loads

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

Uniform Load : D = 0.270, L = 0.340, S = 0.1250 , Tributary Width = 1.0 ft
Uniform Load : D = 0.3563, S = 0.5938 k/ft, Extent = 0.0 -->> 1.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.230, S = 1.950, W = -1.375, E = -2.865 k @ 11.0 ft
Point Load : D = 1.660, S = 2.660, W = 0.60, E = 1.220 k @ 1.0 ft
Uniform Load : D = 0.0450, S = 0.0750 k/ft, Extent = 11.0 -->> 12.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.794** : 1	Maximum Shear Stress Ratio	=	**0.740** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	2,122.07 psi		=	367.23 psi
	=	2,673.75 psi		=	496.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+1.105D+0.750L+0.750S-1.575E
Location of maximum on span	=	5.825 ft	Location of maximum on span	=	11.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.211 in	Ratio =	683 >=360
Max Upward Transient Deflection	-0.038 in	Ratio =	3798 >=360
Max Downward Total Deflection	0.541 in	Ratio =	266 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.522	7.941
Overall MINimum	-0.880	2.525
D Only	3.587	2.944
+D+L	5.627	4.984
+D+S	7.511	5.800
+D+0.750L	5.117	4.474
+D+0.750L+0.750S	8.060	6.616
+D+0.60W	3.849	2.218
+D-0.60W	3.326	3.670
+D+0.70E	4.203	1.177
+D-0.70E	2.972	4.711

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Wood Beam

DESCRIPTION: 8a. Rim Beam at Dining Rm

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.450W	5.313	3.929
+D+0.750L-0.450W	4.922	5.018
+D+0.750L+0.750S+0.450W	8.256	6.071
+D+0.750L+0.750S-0.450W	7.864	7.160
+D+0.750L+0.750S+0.5250E	8.522	5.290
+D+0.750L+0.750S-0.5250E	7.598	7.941
+0.60D+0.60W	2.414	1.040
+0.60D-0.60W	1.891	2.493
+0.60D+0.70E	2.768	-0.001
+0.60D-0.70E	1.537	3.533
L Only	2.040	2.040
S Only	3.923	2.856
W Only	0.435	-1.210
-W	-0.435	1.210
E Only	0.880	-2.525
E Only * -1.0	-0.880	2.525

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Wood Beam

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DESCRIPTION: 9. Covered Porch Roof Beam at Dining Rm

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

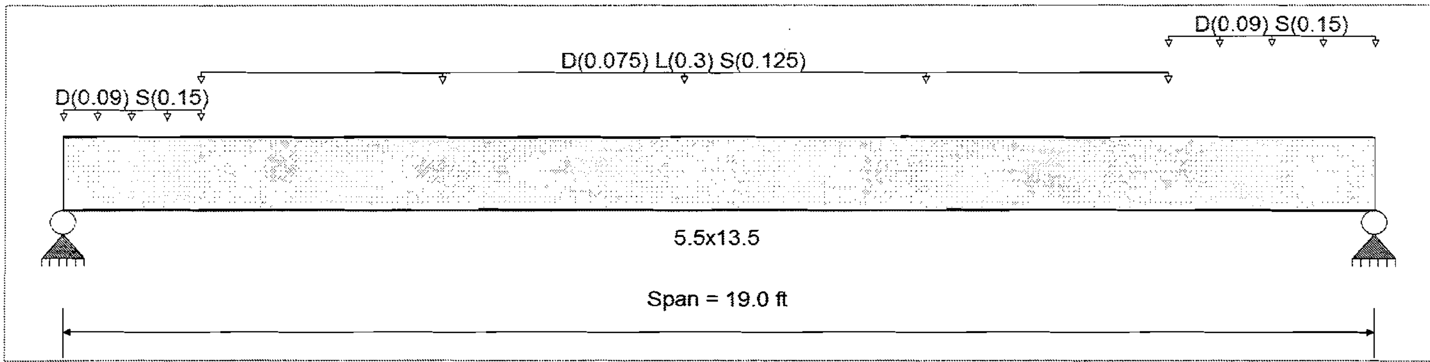
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.090, S = 0.150 k/ft, Extent = 0.0 -->> 2.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.0750, L = 0.30, S = 0.1250 k/ft, Extent = 2.0 -->> 16.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.090, S = 0.150 k/ft, Extent = 16.0 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.483** 1	Maximum Shear Stress Ratio	=	**0.218** : 1
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	1,149.08 psi		=	57.68 psi
	=	2,378.90 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.431 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.399 in	Ratio =	571 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.548 in	Ratio =	415 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.333	3.192
Overall MINimum	1.241	1.259
D Only	0.744	0.756
+D+L	2.955	2.745
+D+S	1.985	2.015
+D+0.750L	2.402	2.248
+D+0.750L+0.750S	3.333	3.192
+0.60D	0.447	0.453
L Only	2.211	1.989
S Only	1.241	1.259

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Wood Beam

Lic. # : KW-06008232

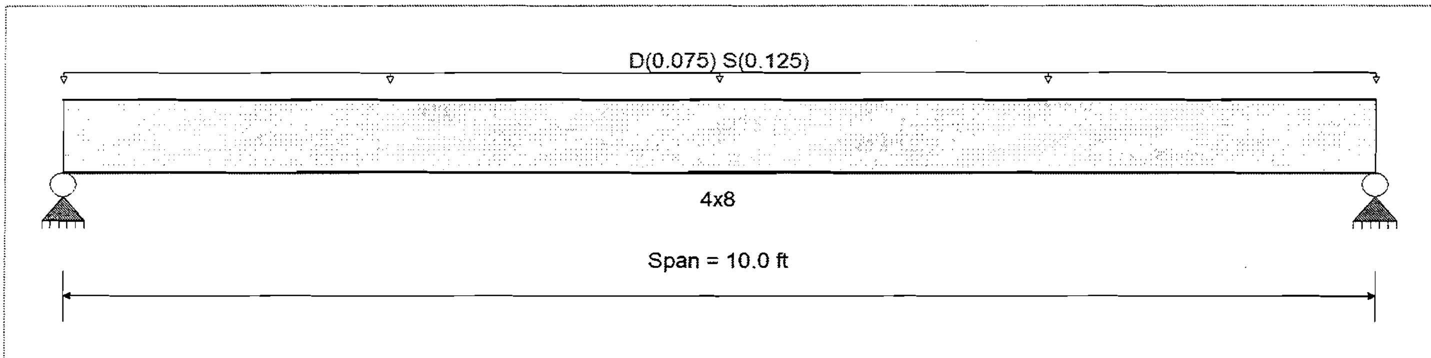
DESCRIPTION: 10. Header at Den

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: DouglasFir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0750, S = 0.1250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.727: 1	Maximum Shear Stress Ratio	=	0.252 : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	978.43 psi		=	52.21 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.159 in	Ratio =	754 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.255 in	Ratio =	471 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.000	1.000
Overall MINimum	0.625	0.625
D Only	0.375	0.375
+D+L	0.375	0.375
+D+S	1.000	1.000
+D+0.750L	0.375	0.375
+D+0.750L+0.750S	0.844	0.844
+0.60D	0.225	0.225
S Only	0.625	0.625

MYERS ENGINEERING
Mark Myers, PE
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3206 50th St. Ct. NW, Ste. 210-B
Gig Harbor, WA 98335

Wood Beam

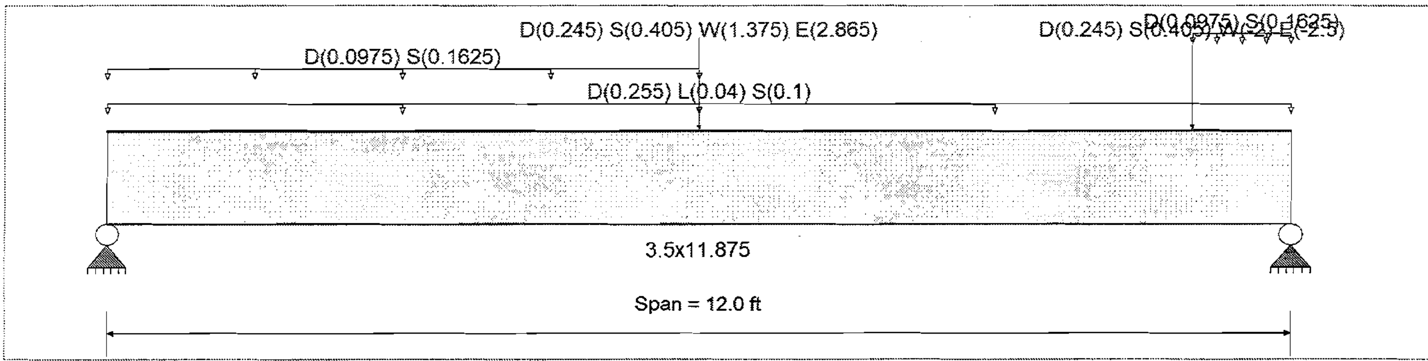
DESCRIPTION: 11. Rim beam over Den

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,050.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.2550, L = 0.040, S = 0.10 , Tributary Width = 1.0 ft
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 0.0 -->> 6.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.09750, S = 0.1625 k/ft, Extent = 11.0 -->> 12.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.2450, S = 0.4050, W = 1.375, E = 2.865 k @ 6.0 ft
Point Load : D = 0.2450, S = 0.4050, W = -2.0, E = -2.50 k @ 11.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.892** 1	Maximum Shear Stress Ratio	=	**0.378** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	3,319.67 psi		=	187.59 psi
	=	3,720.00 psi		=	496.00 psi
Load Combination		+1.155D+2.10E	Load Combination		+1.155D+2.10E
Location of maximum on span	=	6.000 ft	Location of maximum on span	=	10.993 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.186 in	Ratio =	774 >=360
Max Upward Transient Deflection	-0.186 in	Ratio =	774 >=360
Max Downward Total Deflection	0.448 in	Ratio =	321 >=240
Max Upward Total Deflection	-0.002 in	Ratio =	89526 >=240

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.119	3.992
Overall MINimum	-1.224	0.859
D Only	2.116	2.117
+D+L	2.356	2.357
+D+S	3.690	3.690
+D+0.750L	2.296	2.297
+D+0.750L+0.750S	3.476	3.477
+D+0.60W	2.428	1.429
+D-0.60W	1.803	2.804
+D+0.70E	2.973	1.515
+D-0.70E	1.259	2.718

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DESCRIPTION: 11. Rim beam over Den

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.450W	2.530	1.781
+D+0.750L-0.450W	2.061	2.812
+D+0.750L+0.750S+0.450W	3.711	2.961
+D+0.750L+0.750S-0.450W	3.242	3.992
+D+0.750L+0.750S+0.5250E	4.119	3.026
+D+0.750L+0.750S-0.5250E	2.834	3.928
+0.60D+0.60W	1.582	0.583
+0.60D-0.60W	0.957	1.958
+0.60D+0.70E	2.126	0.669
+0.60D-0.70E	0.413	1.871
L Only	0.240	0.240
S Only	1.574	1.573
W Only	0.521	-1.146
-W	-0.521	1.146
E Only	1.224	-0.859
E Only * -1.0	-1.224	0.859

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Wood Beam

DESCRIPTION: 12.Header at Guest Rm

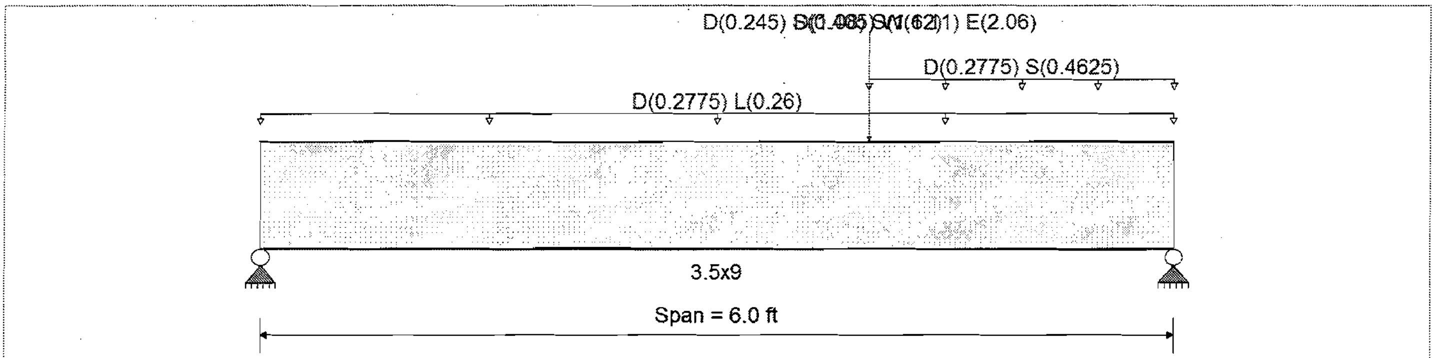
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1850 psi	Ebend- xx	1800 ksi
		Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Wood Species	: DF/DF	Fc - Perp	650 psi	Ebend- yy	1600 ksi
Wood Grade	: 24F-V4	Fv	265 psi	Eminbend - yy	850 ksi
		Ft	1100 psi	Density	31.21 pcf

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



Applied Loads

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

Uniform Load : D = 0.2775, L = 0.260 , Tributary Width = 1.0 ft
Uniform Load : D = 0.2775, S = 0.4625 k/ft, Extent = 4.0 -->> 6.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.2450, S = 0.4050, W = 1.110, E = 2.060 k @ 4.0 ft
Point Load : D = 1.080, S = 1.620 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.740: 1**	Maximum Shear Stress Ratio	=	**0.663 : 1**
Section used for this span		**3.5x9**	Section used for this span		**3.5x9**
	=	2,842.06psi		=	281.20 psi
	=	3,840.00psi		=	424.00 psi
Load Combination		+1.116D+0.750L+0.750S+1.575E	Load Combination		+1.116D+0.750L+0.750S+1.575E
Location of maximum on span	=	3.985ft	Location of maximum on span	=	5.255 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.044 in	Ratio =	1619 >=480
Max Upward Transient Deflection	-0.036 in	Ratio =	1989 >=480
Max Downward Total Deflection	0.117 in	Ratio =	615 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.934	5.075
Overall MINimum	-0.687	-1.373
D Only	1.367	2.178
+D+L	2.147	2.958
+D+S	2.196	4.299
+D+0.750L	1.952	2.763
+D+0.750L+0.750S	2.574	4.354
+D+0.60W	1.589	2.622
+D-0.60W	1.145	1.734
+D+0.70E	1.847	3.140
+D-0.70E	0.886	1.217
+D+0.750L+0.450W	2.118	3.096

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DESCRIPTION: 12.Header at Guest Rm

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L-0.450W	1.785	2.430
+D+0.750L+0.750S+0.450W	2.740	4.687
+D+0.750L+0.750S-0.450W	2.407	4.021
+D+0.750L+0.750S+0.5250E	2.934	5.075
+D+0.750L+0.750S-0.5250E	2.213	3.633
+0.60D+0.60W	1.042	1.751
+0.60D-0.60W	0.598	0.863
+0.60D+0.70E	1.301	2.268
+0.60D-0.70E	0.339	0.346
L Only	0.780	0.780
S Only	0.829	2.121
W Only	0.370	0.740
-W	-0.370	-0.740
E Only	0.687	1.373
E Only * -1.0	-0.687	-1.373

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Wood Beam

Lic. # : KW-06008232

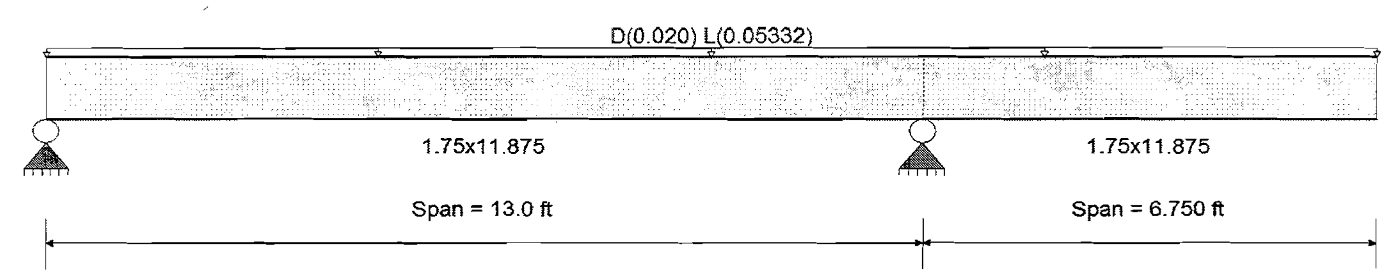
DESCRIPTION: 13. Cantilever Joists

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2325 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2325 psi	Ebend- xx	1550 ksi
		Fc - Prll	2050 psi	Eminbend - xx	787.815 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310 psi		
		Ft	1070 psi	Density	45.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.202** 1	Maximum Shear Stress Ratio	=	**0.125** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	487.30 psi		=	38.67 psi
	=	2,418.00 psi		=	310.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	12.056 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.225 in	Ratio =	718 >=480
Max Upward Transient Deflection	-0.150 in	Ratio =	1076 >=480
Max Downward Total Deflection	0.253 in	Ratio =	638 >=360
Max Upward Total Deflection	-0.122 in	Ratio =	1322 >=360

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.442	1.100	
Overall MINimum	0.253	0.800	
+D+H	0.095	0.300	
+D+L+H, LL Comb Run (*L)	0.001	0.753	
+D+L+H, LL Comb Run (L*)	0.442	0.647	
+D+L+H, LL Comb Run (LL)	0.348	1.100	
+D+Lr+H, LL Comb Run (*L)	0.095	0.300	
+D+Lr+H, LL Comb Run (L*)	0.095	0.300	
+D+Lr+H, LL Comb Run (LL)	0.095	0.300	
+D+S+H	0.095	0.300	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.025	0.640	
+D+0.750Lr+0.750L+H, LL Comb Run (L	0.355	0.560	
+D+0.750Lr+0.750L+H, LL Comb Run (L	0.285	0.900	
+D+0.750L+0.750S+H, LL Comb Run (*L	0.025	0.640	

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DESCRIPTION: 13. Cantilever Joists

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750L+0.750S+H, LL Comb Run (L*	0.355	0.560	
+D+0.750L+0.750S+H, LL Comb Run (LL	0.285	0.900	
+D+0.60W+H	0.095	0.300	
+D-0.60W+H	0.095	0.300	
+D+0.70E+H	0.095	0.300	
+D-0.70E+H	0.095	0.300	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.025	0.640	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.355	0.560	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.285	0.900	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	0.025	0.640	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	0.355	0.560	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	0.285	0.900	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.025	0.640	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.355	0.560	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.285	0.900	
+D+0.750L+0.750S-0.450W+H, LL Comb	0.025	0.640	
+D+0.750L+0.750S-0.450W+H, LL Comb	0.355	0.560	
+D+0.750L+0.750S-0.450W+H, LL Comb	0.285	0.900	
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.025	0.640	
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.355	0.560	
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.285	0.900	
+D+0.750L+0.750S-0.5250E+H, LL Comb	0.025	0.640	
+D+0.750L+0.750S-0.5250E+H, LL Comb	0.355	0.560	
+D+0.750L+0.750S-0.5250E+H, LL Comb	0.285	0.900	
+0.60D+0.60W+0.60H	0.057	0.180	
+0.60D-0.60W+0.60H	0.057	0.180	
+0.60D+0.70E+0.60H	0.057	0.180	
+0.60D-0.70E+0.60H	0.057	0.180	
D Only	0.095	0.300	
L Only, LL Comb Run (*L)	-0.093	0.453	
L Only, LL Comb Run (L*)	0.347	0.347	
L Only, LL Comb Run (LL)	0.253	0.800	
H Only			

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Wood Beam

File: Masin Residence.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

DESCRIPTION: 14. Rim beam at top of stair

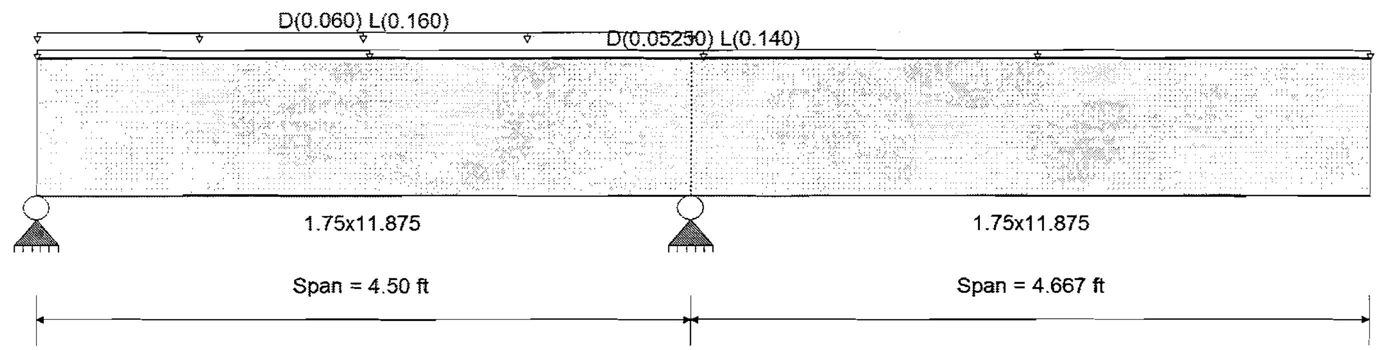
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,050.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf

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



Applied Loads

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

Loads on all spans...
Uniform Load on ALL spans : D = 0.05250, L = 0.140 k/ft
Partial Length Uniform Load : D = 0.060, L = 0.160 k/ft, Extent = 0.0 -->> 4.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.263 : 1**	Maximum Shear Stress Ratio	=	**0.230 : 1**
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	611.65 psi		=	71.43 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	3.520 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.086 in	Ratio =	1294 >=360
Max Upward Transient Deflection	-0.024 in	Ratio =	4614 >=360
Max Downward Total Deflection	0.110 in	Ratio =	1018 >=240
Max Upward Total Deflection	-0.010 in	Ratio =	5478 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.801	2.292	
Overall MINimum	0.336	1.667	
+D+H	0.126	0.625	
+D+L+H, LL Comb Run (*L)	-0.213	1.617	
+D+L+H, LL Comb Run (L*)	0.801	1.300	
+D+L+H, LL Comb Run (LL)	0.462	2.292	
+D+Lr+H, LL Comb Run (*L)	0.126	0.625	
+D+Lr+H, LL Comb Run (L*)	0.126	0.625	
+D+Lr+H, LL Comb Run (LL)	0.126	0.625	
+D+S+H	0.126	0.625	
+D+0.750Lr+0.750L+H, LL Comb Run (*	-0.128	1.369	
+D+0.750Lr+0.750L+H, LL Comb Run (L	0.632	1.131	
+D+0.750Lr+0.750L+H, LL Comb Run (L	0.378	1.876	

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DESCRIPTION: 14. Rim beam at top of stair

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750L+0.750S+H, LL Comb Run (*L	-0.128	1.369	
+D+0.750L+0.750S+H, LL Comb Run (L*	0.632	1.131	
+D+0.750L+0.750S+H, LL Comb Run (LL	0.378	1.876	
+D+0.60W+H	0.126	0.625	
+D-0.60W+H	0.126	0.625	
+D+0.70E+H	0.126	0.625	
+D-0.70E+H	0.126	0.625	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	-0.128	1.369	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.632	1.131	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.378	1.876	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	-0.128	1.369	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	0.632	1.131	
+D+0.750Lr+0.750L-0.450W+H, LL Comb	0.378	1.876	
+D+0.750L+0.750S+0.450W+H, LL Comb	-0.128	1.369	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.632	1.131	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.378	1.876	
+D+0.750L+0.750S-0.450W+H, LL Comb	-0.128	1.369	
+D+0.750L+0.750S-0.450W+H, LL Comb	0.632	1.131	
+D+0.750L+0.750S-0.450W+H, LL Comb	0.378	1.876	
+D+0.750L+0.750S+0.5250E+H, LL Comb	-0.128	1.369	
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.632	1.131	
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.378	1.876	
+D+0.750L+0.750S-0.5250E+H, LL Comb	-0.128	1.369	
+D+0.750L+0.750S-0.5250E+H, LL Comb	0.632	1.131	
+D+0.750L+0.750S-0.5250E+H, LL Comb	0.378	1.876	
+0.60D+0.60W+0.60H	0.076	0.375	
+0.60D-0.60W+0.60H	0.076	0.375	
+0.60D+0.70E+0.60H	0.076	0.375	
+0.60D-0.70E+0.60H	0.076	0.375	
D Only	0.126	0.625	
L Only, LL Comb Run (*L)	-0.339	0.992	
L Only, LL Comb Run (L*)	0.675	0.675	
L Only, LL Comb Run (LL)	0.336	1.667	
H Only			

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Wood Beam

File: Masin Residence.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

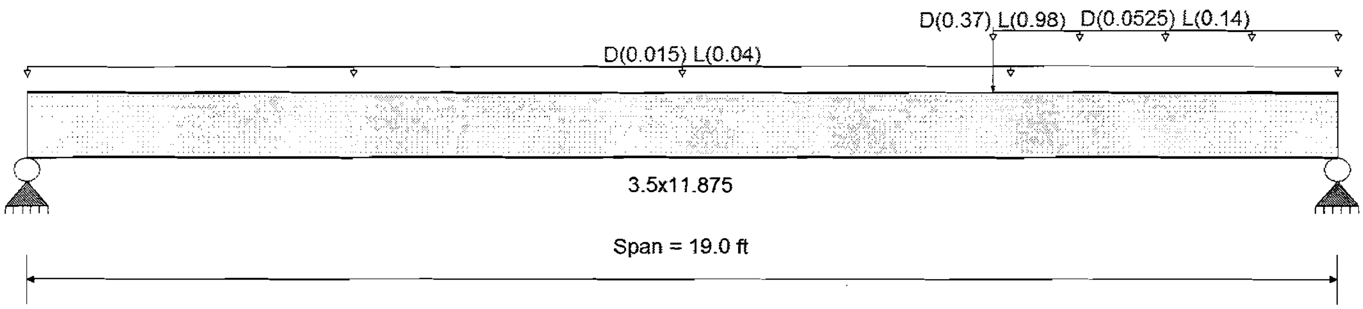
DESCRIPTION: 15. Floor beam at Master Shower

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2600 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2600 psi	Ebend- xx	1900 ksi
		Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285 psi		
		Ft	1555 psi	Density	42.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.370, L = 0.980 k @ 14.0 ft
Uniform Load : D = 0.05250, L = 0.140 k/ft, Extent = 14.0 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.486**: 1	Maximum Shear Stress Ratio	=	**0.268** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	1,263.84psi		=	76.25 psi
	=	2,600.00psi		=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	14.007ft	Location of maximum on span	=	18.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.388 in	Ratio =	587 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.534 in	Ratio =	426 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.004	2.353
Overall MINimum	0.730	1.710
D Only	0.274	0.643
+D+L	1.004	2.353
+D+S	0.274	0.643
+D+0.750L	0.822	1.926
+D+0.750L+0.750S	0.822	1.926
+0.60D	0.165	0.386
L Only	0.730	1.710
S Only		

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Wood Beam

DESCRIPTION: 16. Rim beam at Grid 4

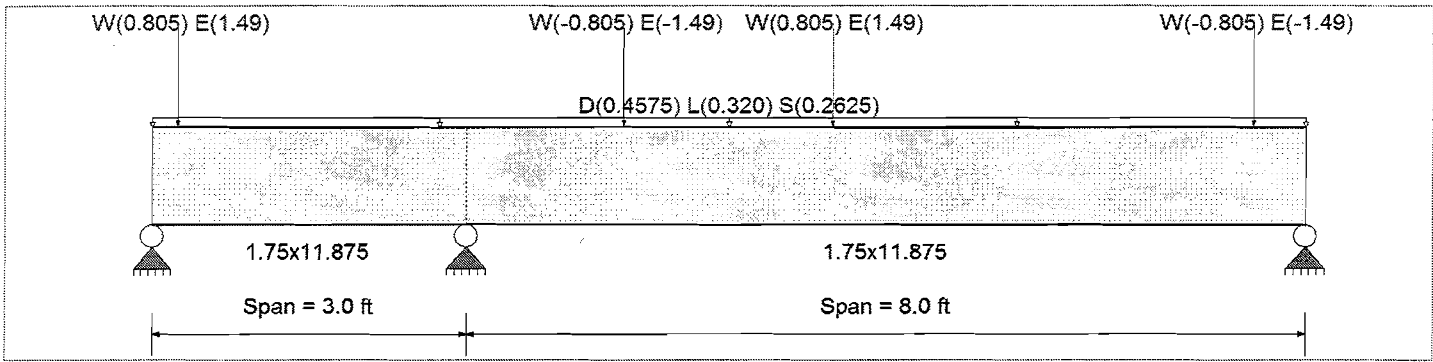
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,050.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf

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



Applied Loads

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

Loads on all spans...
Uniform Load on ALL spans : D = 0.4575, L = 0.320, S = 0.2625 k/ft
Load for Span Number 1
Point Load : W = 0.8050, E = 1.490 k @ 0.250 ft
Load for Span Number 2
Point Load : W = -0.8050, E = -1.490 k @ 1.50 ft
Point Load : W = 0.8050, E = 1.490 k @ 3.50 ft
Point Load : W = -0.8050, E = -1.490 k @ 7.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.598** 1	Maximum Shear Stress Ratio	=	**0.690** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,598.28 psi		=	342.01 psi
	=	2,673.75 psi		=	496.00 psi
Load Combination		+D+0.750L+0.750S+H, LL Comb Run (L	Load Combination		+1.116D+0.750L+0.750S+1.575E+H, LL
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	1.520 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.045 in	Ratio = 2150 >=360
Max Upward Transient Deflection	-0.014 in	Ratio = 6889 >=360
Max Downward Total Deflection	0.128 in	Ratio = 747 >=240
Max Upward Total Deflection	-0.011 in	Ratio = 3160 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-1.493	7.549	3.456
Overall MINimum	-1.282	0.227	1.056
+D+H	-0.248	3.801	1.480
+D+L+H, LL Comb Run (*L)	-0.868	5.934	2.527
+D+L+H, LL Comb Run (L*)	0.199	4.326	1.467
+D+L+H, LL Comb Run (LL)	-0.421	6.459	2.515
+D+Lr+H, LL Comb Run (*L)	-0.248	3.801	1.480
+D+Lr+H, LL Comb Run (L*)	-0.248	3.801	1.480

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 16. Rim beam at Grid 4

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+Lr+H, LL Comb Run (LL)	-0.248	3.801	1.480
+D+S+H	-0.390	5.981	2.329
+D+0.750Lr+0.750L+H, LL Comb Run (*	-0.713	5.401	2.265
+D+0.750Lr+0.750L+H, LL Comb Run (L	0.088	4.194	1.471
+D+0.750Lr+0.750L+H, LL Comb Run (L	-0.378	5.794	2.256
+D+0.750L+0.750S+H, LL Comb Run (*L	-0.820	7.036	2.902
+D+0.750L+0.750S+H, LL Comb Run (L*	-0.019	5.830	2.107
+D+0.750L+0.750S+H, LL Comb Run (LL	-0.484	7.430	2.893
+D+0.60W+H	0.168	3.727	1.138
+D-0.60W+H	-0.664	3.874	1.822
+D+0.70E+H	0.650	3.642	0.741
+D-0.70E+H	-1.145	3.959	2.219
+D+0.750Lr+0.750L+0.450W+H, LL Comb	-0.402	5.345	2.009
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.399	4.139	1.214
+D+0.750Lr+0.750L+0.450W+H, LL Comb	-0.066	5.739	1.999
+D+0.750Lr+0.750L-0.450W+H, LL Comb	-1.025	5.456	2.522
+D+0.750Lr+0.750L-0.450W+H, LL Comb	-0.224	4.249	1.727
+D+0.750Lr+0.750L-0.450W+H, LL Comb	-0.690	5.849	2.513
+D+0.750L+0.750S+0.450W+H, LL Comb	-0.508	6.981	2.645
+D+0.750L+0.750S+0.450W+H, LL Comb	0.293	5.775	1.851
+D+0.750L+0.750S+0.450W+H, LL Comb	-0.173	7.375	2.636
+D+0.750L+0.750S-0.450W+H, LL Comb	-1.132	7.091	3.159
+D+0.750L+0.750S-0.450W+H, LL Comb	-0.331	5.885	2.364
+D+0.750L+0.750S-0.450W+H, LL Comb	-0.796	7.485	3.149
+D+0.750L+0.750S+0.5250E+H, LL Comb	-0.147	6.917	2.348
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.654	5.711	1.553
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.189	7.311	2.339
+D+0.750L+0.750S-0.5250E+H, LL Comb	-1.493	7.155	3.456
+D+0.750L+0.750S-0.5250E+H, LL Comb	-0.692	5.949	2.662
+D+0.750L+0.750S-0.5250E+H, LL Comb	-1.158	7.549	3.447
+0.60D+0.60W+0.60H	0.267	2.207	0.546
+0.60D-0.60W+0.60H	-0.564	2.354	1.230
+0.60D+0.70E+0.60H	0.749	2.122	0.149
+0.60D-0.70E+0.60H	-1.046	2.439	1.627
D Only	-0.248	3.801	1.480
L Only, LL Comb Run (*L)	-0.621	2.133	1.047
L Only, LL Comb Run (L*)	0.447	0.525	-0.012
L Only, LL Comb Run (LL)	-0.173	2.658	1.035
S Only	-0.142	2.181	0.849
W Only	0.693	-0.122	-0.570
-W	-0.693	0.122	0.570
E Only	1.282	-0.227	-1.056
E Only * -1.0	-1.282	0.227	1.056
H Only			

MYERS ENGINEERING

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Wood Beam

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DESCRIPTION: 17. Rim Beam at Grid C

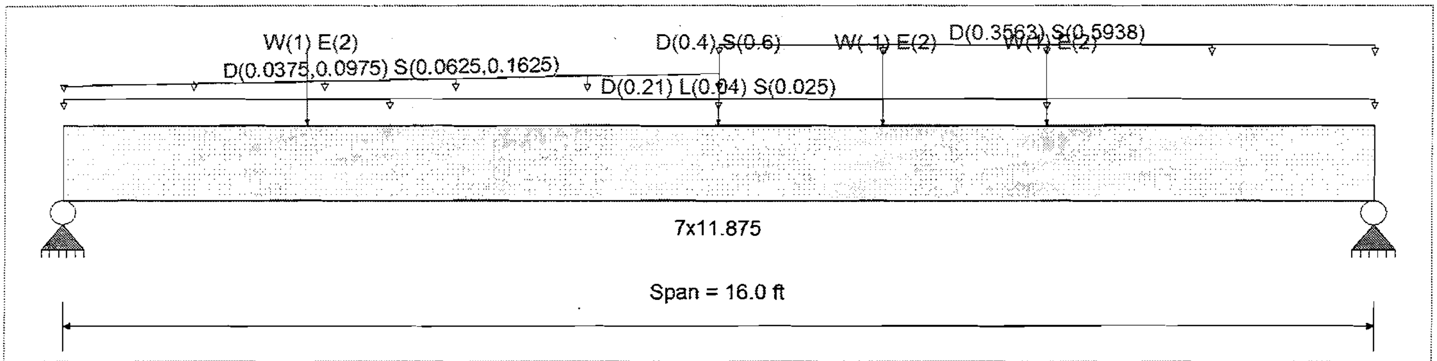
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf

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



Applied Loads

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

Uniform Load : D = 0.210, L = 0.040, S = 0.0250 , Tributary Width = 1.0 ft
Varying Uniform Load : D= 0.03750->0.09750, S= 0.06250->0.1625 k/ft, Extent = 0.0 -->> 8.0 ft, Trib Width = 1.0 ft

Uniform Load : D = 0.3563, S = 0.5938 k/ft, Extent = 8.0 -->> 16.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.40, S = 0.60 k @ 8.0 ft
Point Load : W = 1.0, E = 2.0 k @ 3.0 ft
Point Load : W = -1.0, E = 2.0 k @ 10.0 ft
Point Load : W = 1.0, E = 2.0 k @ 12.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.814**: 1	Maximum Shear Stress Ratio	=	**0.467** : 1
Section used for this span		**7x11.875**	Section used for this span		**7x11.875**
	=	3,777.44 psi		=	216.68 psi
	=	4,640.00 psi		=	464.00 psi
Load Combination		+1.116D+0.750L+0.750S+1.575E	Load Combination		+1.116D+0.750L+0.750S+1.575E
Location of maximum on span	=	9.985 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.339 in	Ratio =	567 >=480
Max Upward Transient Deflection	-0.325 in	Ratio =	590 >=480
Max Downward Total Deflection	0.801 in	Ratio =	239 >=180
Max Upward Total Deflection	-0.016 in	Ratio =	11899 >=180

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.474	9.294
Overall MINimum	-2.875	-3.125
D Only	2.978	4.173
+D+L	3.298	4.493
+D+S	5.307	8.494
+D+0.750L	3.218	4.413
+D+0.750L+0.750S	4.965	7.654
+D+0.60W	3.390	4.360

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DESCRIPTION: 17. Rim Beam at Grid C

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
+D-0.60W	2.565	3.985
+D+0.70E	4.990	6.360
+D-0.70E	0.965	1.985
+D+0.750L+0.450W	3.527	4.553
+D+0.750L-0.450W	2.908	4.272
+D+0.750L+0.750S+0.450W	5.274	7.794
+D+0.750L+0.750S-0.450W	4.655	7.513
+D+0.750L+0.750S+0.5250E	6.474	9.294
+D+0.750L+0.750S-0.5250E	3.455	6.013
+0.60D+0.60W	2.199	2.691
+0.60D-0.60W	1.374	2.316
+0.60D+0.70E	3.799	4.691
+0.60D-0.70E	-0.226	0.316
L Only	0.320	0.320
S Only	2.329	4.321
W Only	0.688	0.313
-W	-0.688	-0.313
E Only	2.875	3.125
E Only * -1.0	-2.875	-3.125

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Wood Beam

File: Masin Residence.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

DESCRIPTION: 18. Garage Door Header

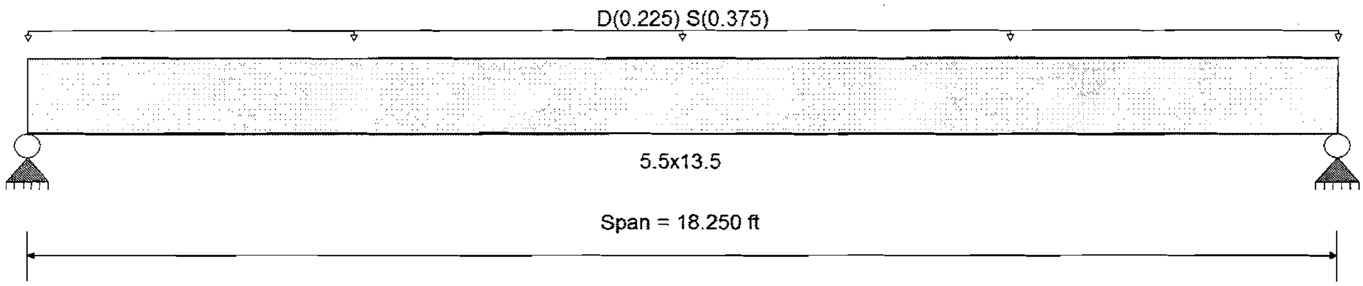
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Uniform Load : D = 0.2250, S = 0.3750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.653** : 1	Maximum Shear Stress Ratio	=	**0.321** : 1
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	1,794.28 psi		=	97.69 psi
	=	2,746.77 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.464 in	Ratio =	472 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.742 in	Ratio =	295 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.475	5.475
Overall MINimum	3.422	3.422
D Only	2.053	2.053
+D+L	2.053	2.053
+D+S	5.475	5.475
+D+0.750L	2.053	2.053
+D+0.750L+0.750S	4.620	4.620
+0.60D	1.232	1.232
S Only	3.422	3.422

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Wood Beam

File: Masin Residence.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

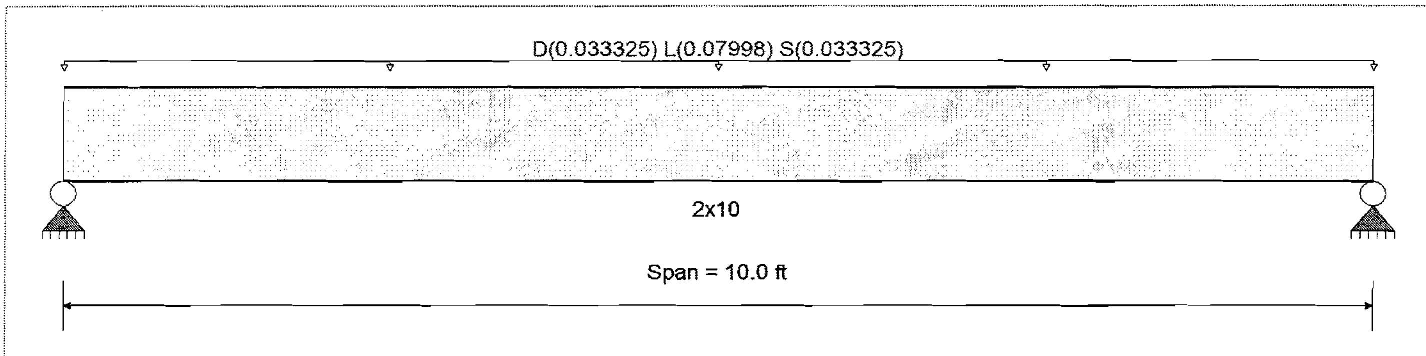
DESCRIPTION: 19. Lower Deck Joists

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Uniform Load : D = 0.0250, L = 0.060, S = 0.0250 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.924 : 1**	Maximum Shear Stress Ratio	=	**0.432 : 1**
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	794.54 psi		=	51.86 psi
	=	860.20 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.148 in	Ratio =	810 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.219 in	Ratio =	547 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.592	0.592
Overall MINimum	0.167	0.167
D Only	0.167	0.167
+D+L	0.567	0.567
+D+S	0.333	0.333
+D+0.750L	0.467	0.467
+D+0.750L+0.750S	0.592	0.592
+0.60D	0.100	0.100
L Only	0.400	0.400
S Only	0.167	0.167

MYERS ENGINEERING

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Wood Beam

DESCRIPTION: 20. Lower Deck/Porch Beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

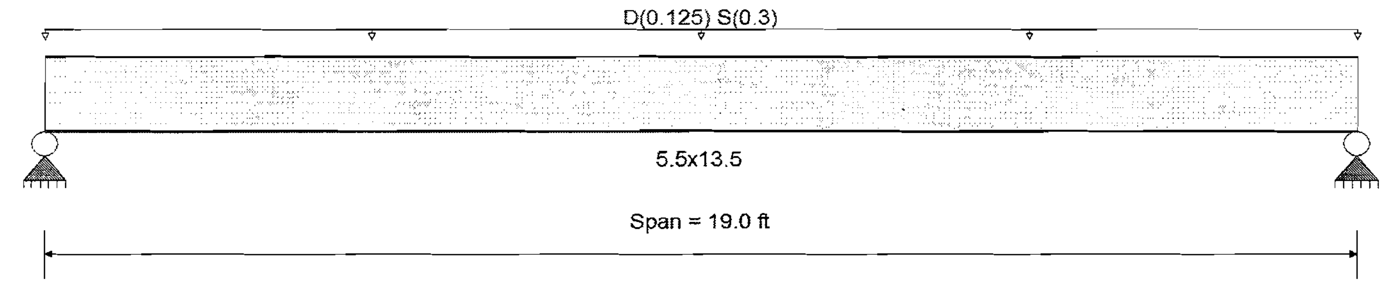
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2400 psi	*E : Modulus of Elasticity*	
Fb -	1850 psi	Ebend- xx	1800 ksi
Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Fc - Perp	650 psi	Ebend- yy	1600 ksi
Fv	265 psi	Eminbend - yy	850 ksi
Ft	1100 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.1250, S = 0.30 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.504: 1	Maximum Shear Stress Ratio	=	0.236 : 1
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	1,377.55psi		=	72.04 psi
	=	2,735.73psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.500ft	Location of maximum on span	=	17.891 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.436 in	Ratio =	523 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.618 in	Ratio =	369 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.038	4.038
Overall MINimum	2.850	2.850
D Only	1.188	1.188
+D+L	1.188	1.188
+D+S	4.038	4.038
+D+0.750L	1.188	1.188
+D+0.750L+0.750S	3.325	3.325
+0.60D	0.713	0.713
S Only	2.850	2.850

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Wood Beam

DESCRIPTION: 20a. Lower Deck/Porch Beam

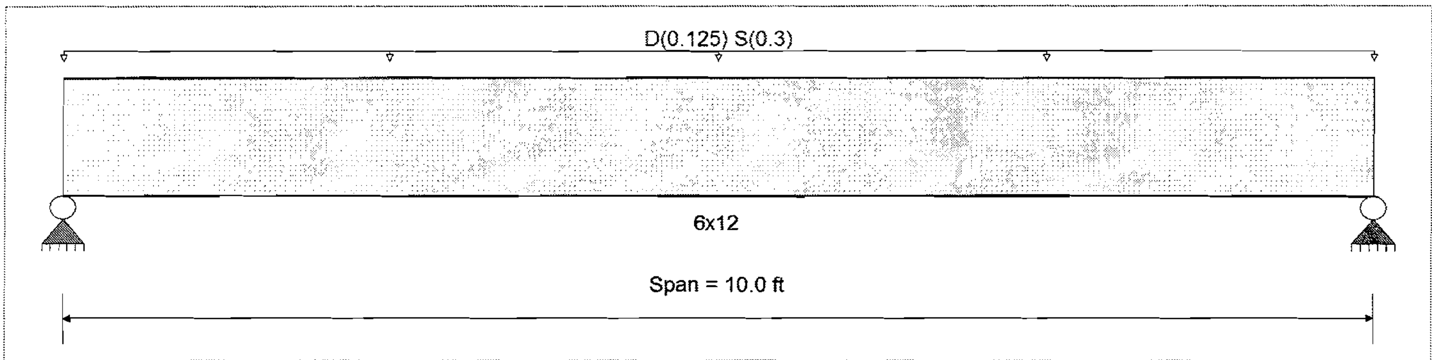
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	675 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	675 psi	Ebend- xx	1100 ksi
		Fc - Prll	500 psi	Eminbend - xx	400 ksi
Wood Species	: Hem-Fir	Fc - Perp	405 psi		
Wood Grade	: No.2	Fv	140 psi		
		Ft	350 psi	Density	26.84 pcf

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



Applied Loads

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

Uniform Load : D = 0.1250, S = 0.30 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.847: 1**	Maximum Shear Stress Ratio	=	**0.317 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	525.86psi		=	40.83 psi
	=	621.00psi		=	128.80 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	1287 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.132 in	Ratio =	908 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.125	2.125
Overall MINimum	1.500	1.500
D Only	0.625	0.625
+D+L	0.625	0.625
+D+S	2.125	2.125
+D+0.750L	0.625	0.625
+D+0.750L+0.750S	1.750	1.750
+0.60D	0.375	0.375
S Only	1.500	1.500

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Wood Beam

DESCRIPTION: 21. Crawl Space beam NOT at bearing wall

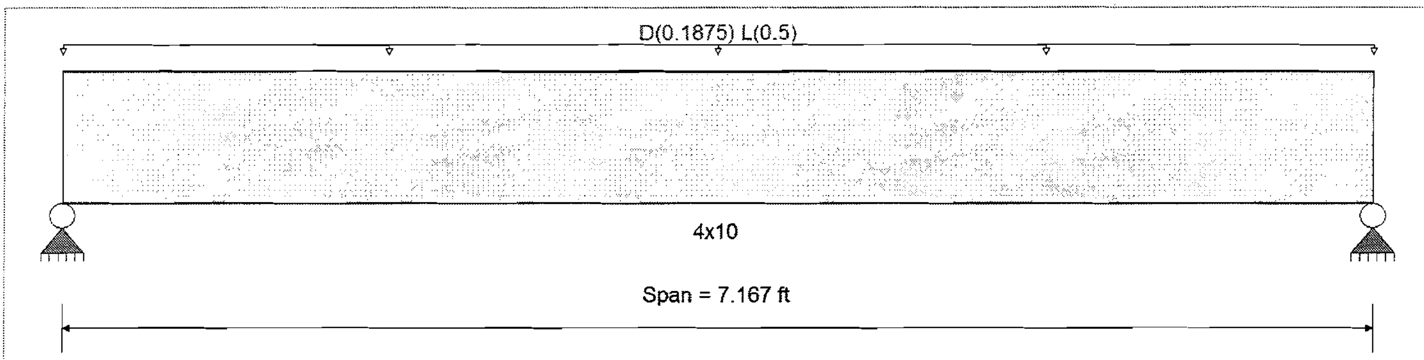
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: DouglasFir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf

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



Applied Loads

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

Uniform Load : D = 0.1875, L = 0.50 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.983 : 1**	Maximum Shear Stress Ratio	=	**0.500 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,061.30 psi		=	89.98 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.584 ft	Location of maximum on span	=	6.408 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.081 in	Ratio =	1063 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.111 in	Ratio =	773 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.464	2.464
Overall MINimum	1.792	1.792
D Only	0.672	0.672
+D+L	2.464	2.464
+D+S	0.672	0.672
+D+0.750L	2.016	2.016
+D+0.750L+0.750S	2.016	2.016
+0.60D	0.403	0.403
L Only	1.792	1.792
S Only		

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Wood Beam

DESCRIPTION: 22. Floor beam under stair rim/cantilever

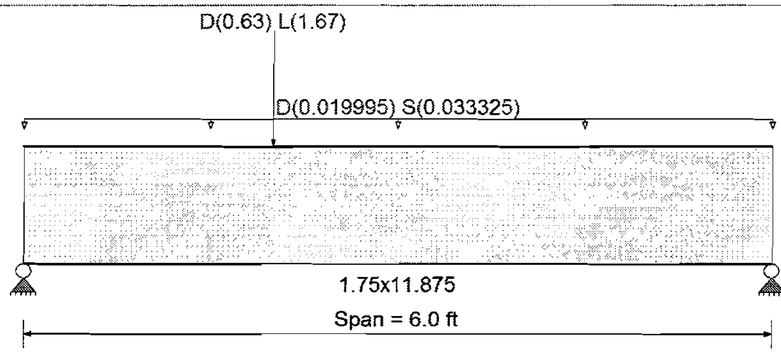
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2325 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2325 psi	Ebend- xx	1550 ksi
		Fc - Prll	2050 psi	Eminbend - xx	787.815 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	800 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310 psi		
		Ft	1070 psi	Density	45.01 pcf

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



Applied Loads

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

Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 1.333 ft
Point Load : D = 0.630, L = 1.670 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.394 1	Maximum Shear Stress Ratio	=	0.366 : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	914.89 psi		=	113.58 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.015 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.030 in	Ratio =	2424 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.042 in	Ratio =	1696 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.593	0.827
Overall MINimum	0.100	0.100
D Only	0.480	0.270
+D+L	1.593	0.827
+D+S	0.580	0.370
+D+0.750L	1.315	0.687
+D+0.750L+0.750S	1.390	0.762
+0.60D	0.288	0.162
L Only	1.113	0.557
S Only	0.100	0.100

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 1008 \cdot psi$

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

$A := t \cdot h$ $A = 30.2 \cdot in^2$

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

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

$C_p = 0.69$

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 659 \cdot psi$

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

$A := t \cdot h$ $A = 30.2 \cdot in^2$

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

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

$C_p = 0.8$

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PROJECT :**Masin Residence**

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

Maximum Load For 3-2x4 HF Stud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf\cdot ft$ $lb := plf\cdot ft$ $H := 10\cdot ft$

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

$E' := 1200000\cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5\cdot in$

$t := 3\cdot 1.5\cdot in$

$A := t\cdot h$ $A = 15.7\cdot in^2$

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

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

$C_p = 0.32$

$F'_c := C_p\cdot F''_c$ $F'_c = 280\cdot psi$ $P_{max} := F'_c\cdot A$ $P_{max} = 4411\cdot lb$ (Maximum post Capacity)

Maximum Load For 2-2x4 HFStud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf\cdot ft$ $lb := plf\cdot ft$ $H := 10\cdot ft$

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

$E' := 1200000\cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5\cdot in$

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

$A := t\cdot h$ $A = 10.5\cdot in^2$

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

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

$C_p = 0.32$

$F'_c := C_p\cdot F''_c$ $F'_c = 280\cdot psi$ $P_{max} := F'_c\cdot A$ $P_{max} = 2941\cdot lb$ (Maximum post Capacity)

Maximum Load For 4x4 HF#2 Treated Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 6.25 \cdot ft$

$F_c := 1040 \cdot psi$ $C_D := 1$ $C_{Fb} := 1$ $C_M := 1$ $C_t := 1$ $C_L := 1$ $C_{Fc} := 1$

$E' := 1235000 \cdot psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc}$ $F''_c = 1040 \cdot psi$

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 807 \cdot psi$

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

4x4 Treated Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

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

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

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

$C_p = 0.6$

$F'_c := C_p \cdot F''_c$ $F'_c = 622 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 7618 \cdot lb$ (Maximum post Capacity)