MYERS ENGINEERING

LATERAL ANALYSIS & GRAVITY CALCULATIONS

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Date: 2021.02.17 16:22:16 -08'00'

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Project: Proposed Residence for American Classic Homes
42xx 89th Avenue Southeast
Mercer Island, WA

February 17, 2021

2018 INTERNATIONAL BUILDING CODE
110 MPH WIND, EXPOSURE B, K_{zt} = 1.40
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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

PROJECT : **42xx 89th AVE SE**

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DESIGN LOADS:

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

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

WOODS :	WOOD TYPE:
JOISTS OR RAFTERS 2X.	HF#2
BEAMS OR HEADERS 4X - 6X OR LARGER	DF#2
LEDGERS AND TOP PLATES	HF#2
STUDS 2X4 OR 2X6	HF 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.

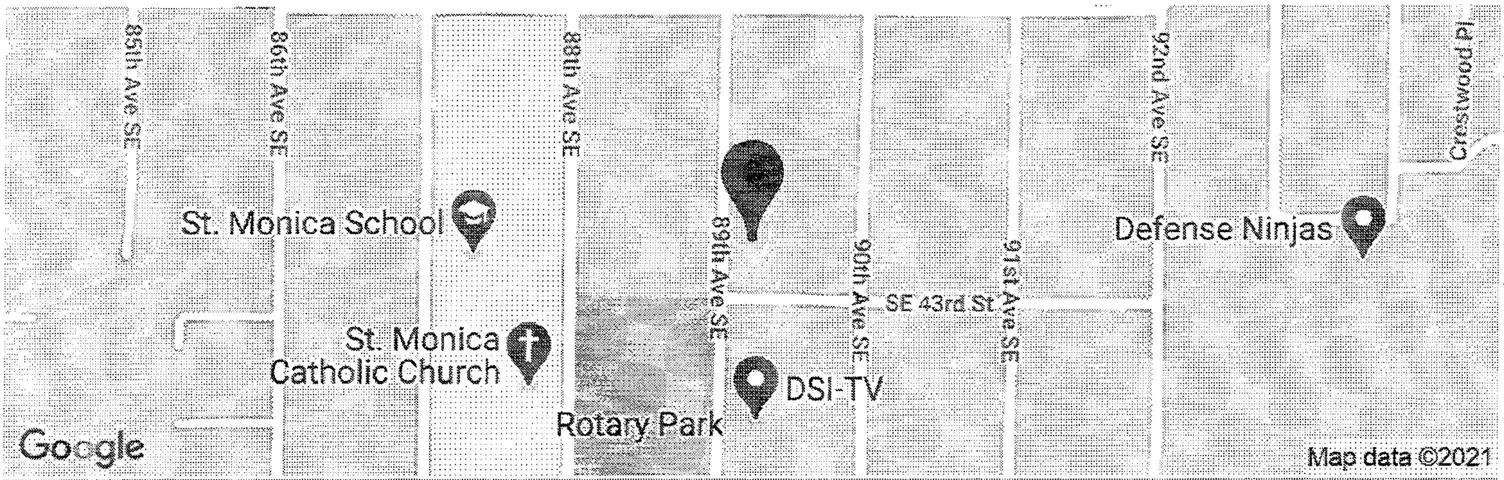
ENGINEERED I-JOISTS

-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

42xx 89th Ave SE

Latitude, Longitude: 47.5695, -122.2199



Date	2/3/2021, 3:31:35 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.419	MCE_R ground motion. (for 0.2 second period)
S_1	0.493	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.702	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.135	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	0.607	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.729	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.419	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.572	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.738	Factored deterministic acceleration value. (0.2 second)
S1RT	0.493	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.549	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.483	Factored deterministic acceleration value. (1.0 second)
PGAd	1.268	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.898	Mapped value of the risk coefficient at a period of 1 s

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)

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

$S_s := 1.419$ $S_1 := 0.493$ $S_{ms} := 1.702$ $S_{m1} := 0.89$

Equation 16-39 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.13$ Equation 16-40 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.59$

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

Roof Slope Adjustment Factor: $S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{7}{12}\right)\right)} = 1.16$ $S_b := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{4}{12}\right)\right)} = 1.05$

Plan Area for Each Level:

$A_1 := 2245 ft^2 \cdot S_a$ (Upper Roof) $A_{2a} := 1841 ft^2$ (Upper Floor) $A_{2b} := 1686 ft^2 \cdot S_b$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(55ft) + 2(55ft)$ (Upper Floor) $P_2 := 2(59ft) + 2(55ft)$ (Main Floor)

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

Weight of Structure at Each Level:

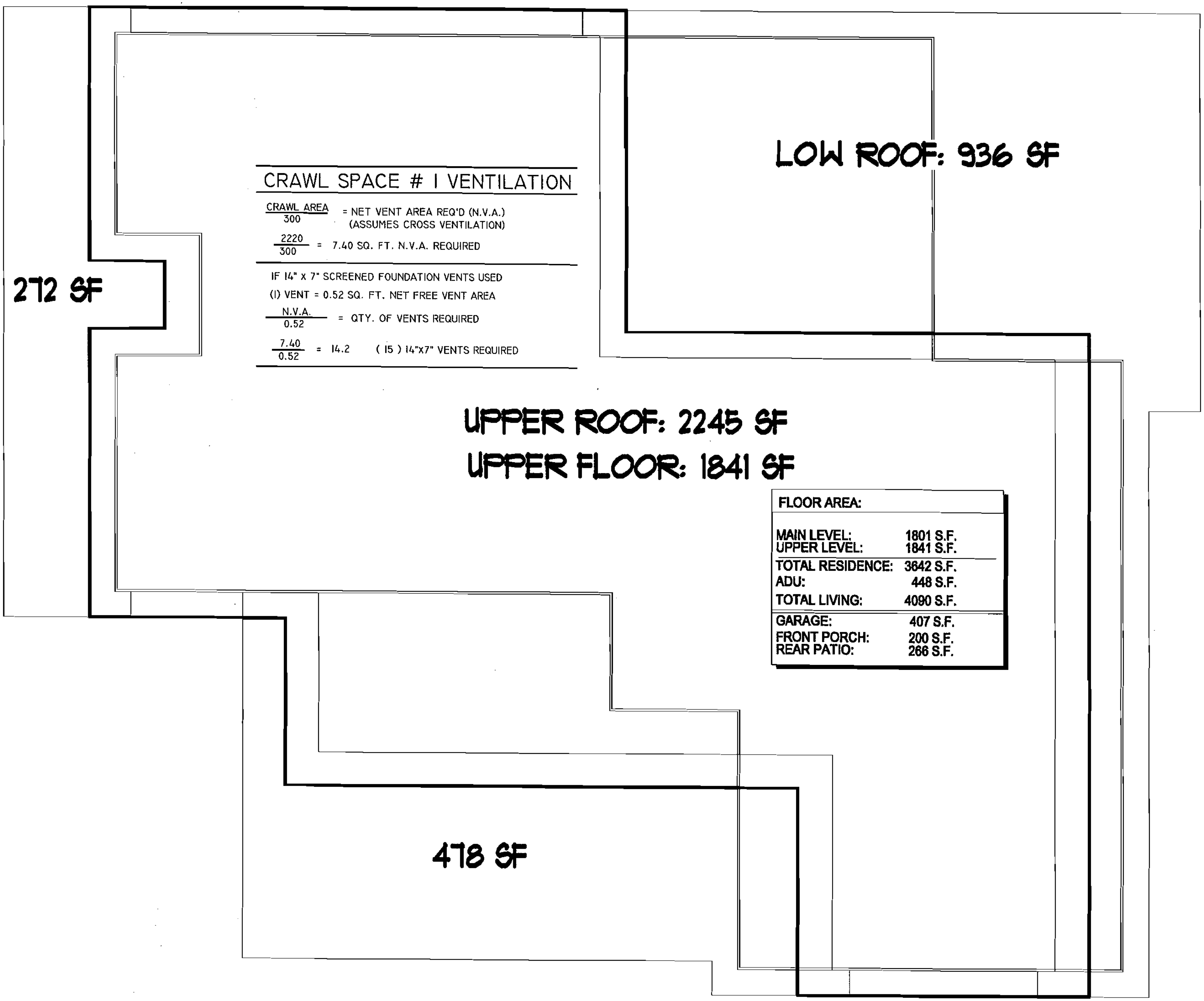
Story Weight at Upper Floor:

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

Story Weight at Main Floor:

$$w_2 := 15 \cdot psf \cdot (A_{2a} + A_{2b}) + 12 \cdot psf \cdot (4.5 \cdot ft \cdot P_1 + 5ft \cdot P_2)$$

$$W := w_1 + w_2 = 130698.67\,lb$$



LOW ROOF: 936 SF
CRAWL SPACE # I VENTILATION
CRAWL AREA / 300 = NET VENT AREA REQ'D (N.V.A.) (ASSUMES CROSS VENTILATION)
2220 / 300 = 7.40 SQ. FT. N.V.A. REQUIRED
IF 14" x 7" SCREENED FOUNDATION VENTS USED
(1) VENT = 0.52 SQ. FT. NET FREE VENT AREA
N.V.A. / 0.52 = QTY. OF VENTS REQUIRED
7.40 / 0.52 = 14.2 (15) 14"x7" VENTS REQUIRED
272 SF
UPPER ROOF: 2245 SF
UPPER FLOOR: 1841 SF
FLOOR AREA:
MAIN LEVEL: 1801 S.F.
UPPER LEVEL: 1841 S.F.
TOTAL RESIDENCE: 3642 S.F.
ADU: 448 S.F.
TOTAL LIVING: 4090 S.F.
GARAGE: 407 S.F.
FRONT PORCH: 200 S.F.
REAR PATIO: 266 S.F.
478 SF

Approximate Fundamental Period, $T_{a:}$

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $\quad h_n := 24$ (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. 22-12)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.42$ (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)

$C_s := \dfrac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.17$ Total Base Shear: $V_E := C_s \cdot W = 22815.3\,\text{lb}$

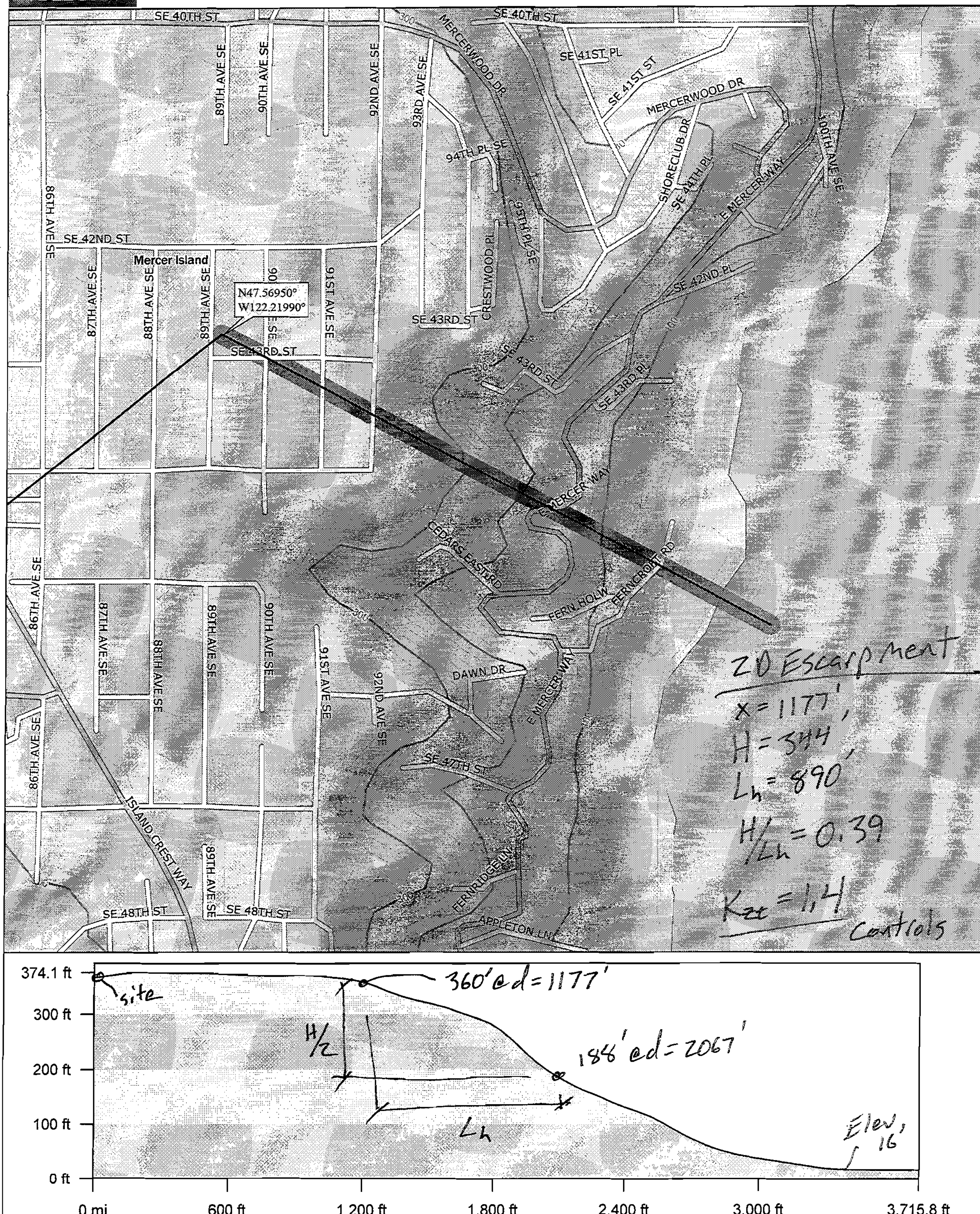
Vertical Shear distribution at each level:

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)

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.56$ $\quad F_1 := C_{v1} \cdot V_E = 12783.52\,\text{lb}$ Story Shear at Upper Floor

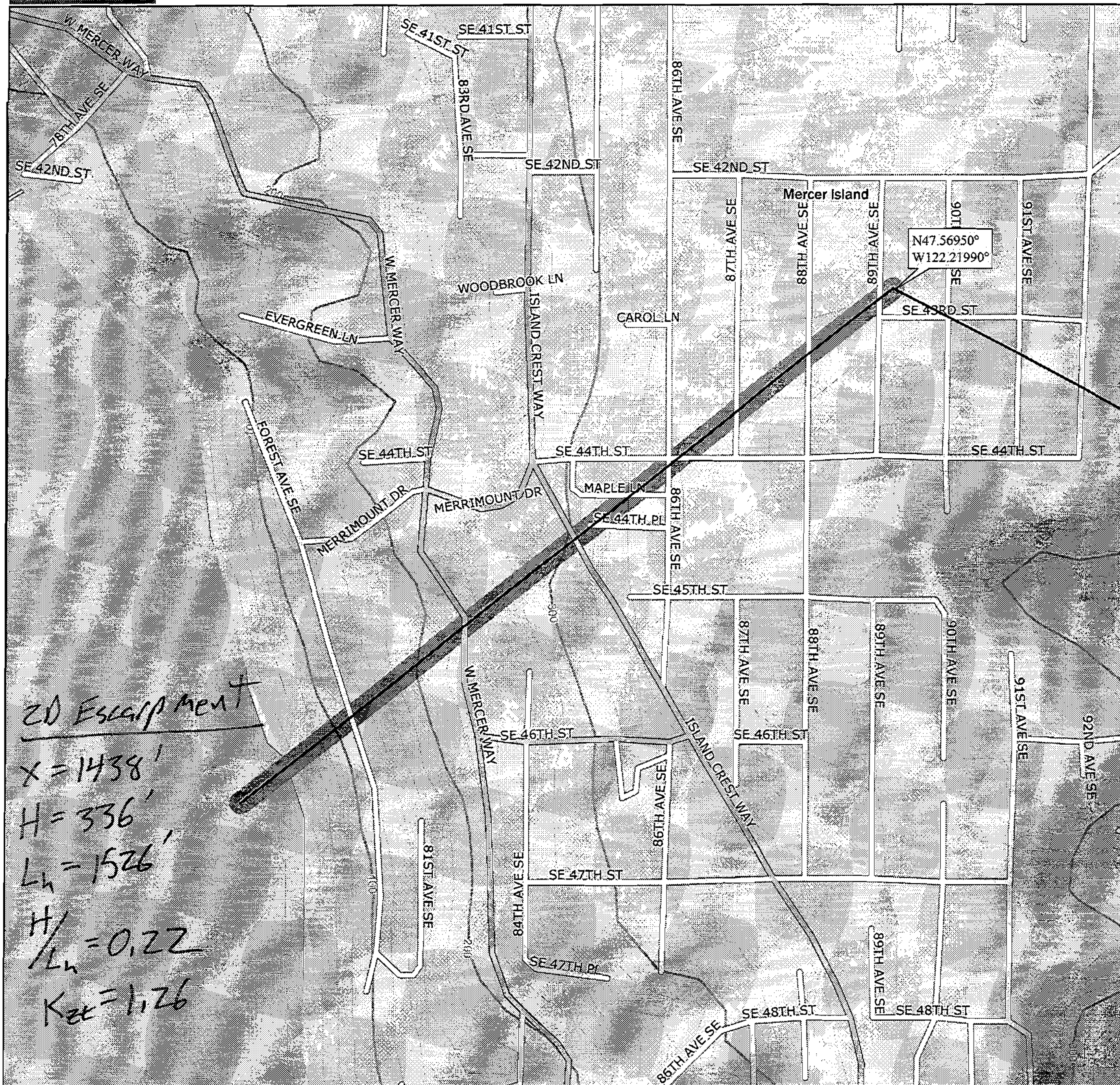
$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.44$ $\quad F_2 := C_{v2} \cdot V_E = 10031.78\,\text{lb}$ Story Shear at Main Floor

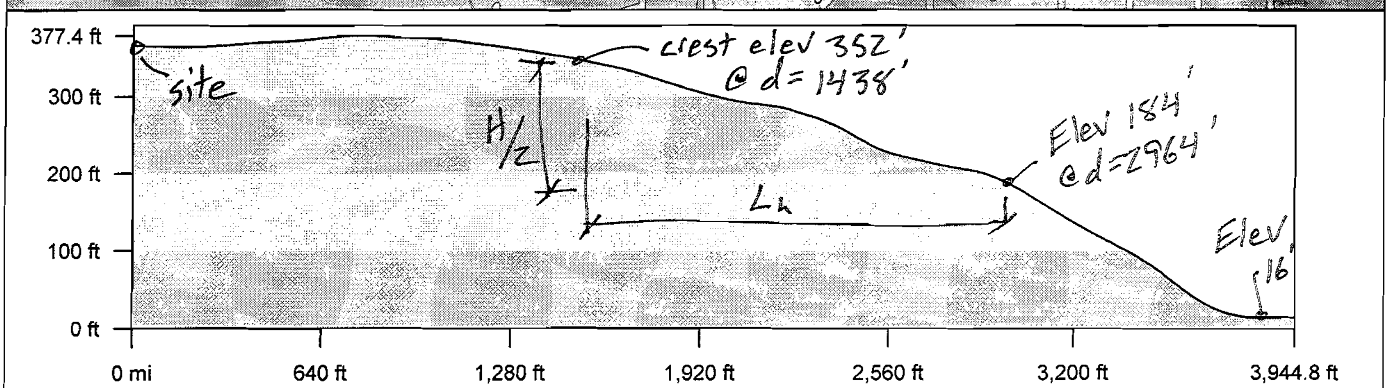


Lin Dist: 3,681.2 ft	Terr Dist: 3,715.8 ft	Elev Gain: -349.0 ft	Avg Grade: 10
Climb Elev: 10.9 ft	Desc Elev: 359.9 ft	Max. Elev: 374.1 ft	Min. Elev: 16.2 ft
Climb Dist: 493.2 ft	Desc Dist: 3,208.0 ft		

TN
MN (14.9°E)

Scale 1 : 9,600
1" = 800.0 ft
Data Zoom 14-4





Lin Dist: 3,914.6 ft	Terr Dist: 3,944.8 ft	Elev Gain: -349.0 ft	Avg Grade: 9
Climb Elev: 15.6 ft	Desc Elev: 364.6 ft	Max. Elev: 377.4 ft	Min. Elev: 15.4 ft
Climb Dist: 706.2 ft	Desc Dist: 3,238.6 ft		

TN
MN (14.9°E)

Scale 1 : 8,800
1" = 733.3 ft
Data Zoom 14-5

WIND DESIGN

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

$V := 110$ 3-Sec Peak Gust (MPH) for Risk Category II (Figure 26.5-1A).

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 24 \cdot ft$ Mean Roof Height as per Sect. 26.2

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

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

$x := 1177ft$ $H := 344ft$ $L_h := 890ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.75\left(\frac{H}{L_h}\right) = 0.29 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.67 \qquad K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.93 \qquad K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1.4$$

$G := 0.85$ Gust Effect Factor (ASCE 7-16 Sect. 26.9.1)

Building is an Enclosed Building as per ASCE 7-16 Sect. 26.10

$GC_{pi} := .18$ +/- Internal Pressure Coefficients (ASCE 7-16 Table 26.11-1)

Velocity Pressure Exposure Coefficient (Table 27.3-1):

$z_g := 1200ft$ $\alpha := 7.0$ (per ASCE 7-16 Table 26.9-1 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)

$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.62 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.57 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.66$$

External Pressure Coefficients w/ Roof Pitch = 7/12 (30 degrees) Front to Back & 7/12 (30 degrees) Side to Side
Taken from Figure 27.4-1

Front to Back:

$L_{fb} := 55ft$ $B_{fb} := 55ft$ $\frac{L_{fb}}{B_{fb}} = 1$ $\frac{h}{L_{fb}} = 0.44$

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

$C_{pf2} := 0.23$ Windward Roof

$C_{pf3} := -.6$ Leeward Roof

$C_{pf4} := -.5$ Leeward Wall

Side to Side:

$L_{ss} := 55ft$ $B_{ss} := 55ft$ $\frac{L_{ss}}{B_{ss}} = 1$ $\frac{h}{L_{ss}} = 0.44$

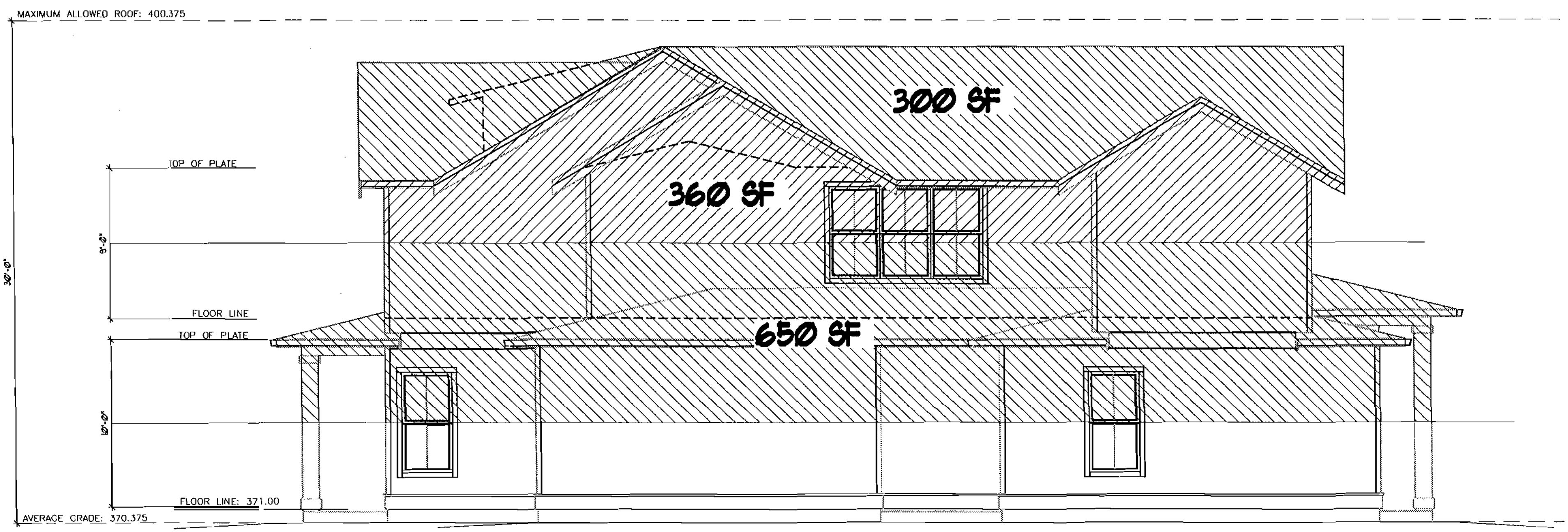
$C_{ps1} := .8$ Windward Wall

$C_{ps2} := 0.23$ Windward Roof

$C_{ps3} := -.6$ Leeward Roof

$C_{ps4} := -.5$ Leeward Wall

FRONT ENTRY (WEST) ELEVATION



RIGHT SIDE (SOUTH) ELEVATION

9

Velocity Pressure (q_z) Evaluated at Height (z) (Equation 23.3-1)

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot V^2 = 22.93$ $\quad q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot V^2 = 21.12$ $\quad q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 = 24.16$

Design Wind Pressures $p = qGC_p - q_i(GC_{pi})$ (Equation 27.4-1) where q_i will conservatively be taken equal to q_h

Windward Roof Front to Back $\quad p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 4.72\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $\quad p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -12.32\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $\quad p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -10.27\,ft^{-2} \cdot lb$

Windward Roof Side to Side $\quad p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 4.72\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $\quad p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -12.32\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $\quad p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -10.27\,ft^{-2} \cdot lb$

Windward Wall Both Directions

$p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 15.59\,ft^{-2} \cdot lb$

$p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 14.36\,ft^{-2} \cdot lb$

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

Check net pressure not less than 16psf at walls & 8psf at roof over projected vertical plane:

$p_{wr1} - p_{lr1} = 17.04\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw1} = 25.86\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw1} = 24.63\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 17.04\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw2} = 25.86\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw2} = 24.63\,ft^{-2} \cdot lb$

<u>Wind Pressure at Upper Roof (Front to Back):</u>

$$V_{1W} := (p_{wr1} - p_{lr1})265ft^2 + (p_{ww1} - p_{lw1}) \cdot 350 \cdot ft^2 = 13566.21\,lb$$

<u>Wind Pressure at 2nd Floor (Front to Back):</u>

$$V_{2W} := (p_{wr1} - p_{lr1})0ft^2 + (p_{ww2} - p_{lw1}) \cdot 595 \cdot ft^2 = 14653.33\,lb$$

<u>Wind Pressure at Upper Roof (Side to Side):</u>

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 300ft^2 + (p_{ww1} - p_{lw2}) \cdot 360ft^2 = 14421.24\,lb$$

<u>Wind Pressure at 2nd Floor (Side to Side):</u>

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 650ft^2 = 16007.84\,lb$$

Determine Component & Cladding loads:

Design Wind Pressures $p = q_h[(GC_p) - (GC_{pi})]$ (Equation 30.3-1)

(GC_p) is given in Figures 30.3-1 thru 30.3-7

(GC_{pi}) is given in Table 26.13-1 (See above)

$GC_{p1in} := 0.9$ $GC_{p2in} := 0.9$ $GC_{p3in} := 0.9$ Figure 30.3-2D (θ = 30 degrees)

$GC_{p1out} := -1.8$ $GC_{p2out} := -2.0$ $GC_{p3out} := -3.2$ $GC_{p2oh} := -2.8$ $GC_{p3oh} := -4.0$

$GC_{p4in} := 1.0$ $GC_{p5in} := 1.0$ Figure 30.3-1

$GC_{p4out} := -1.1$ $GC_{p5out} := -1.4$

$p_1 := q_h \cdot \left[\left(GC_{p1out}\right) - \left(GC_{pi}\right)\right] psf$ $p_1 = -47.83\, ft^{-2} \cdot lb$ (Zone 1)

$p_2 := q_h \cdot \left[\left(GC_{p2out}\right) - \left(GC_{pi}\right)\right] psf$ $p_2 = -52.66\, ft^{-2} \cdot lb$ (Zone 2)

$p_3 := q_h \cdot \left[\left(GC_{p3out}\right) - \left(GC_{pi}\right)\right] psf$ $p_3 = -81.64\, ft^{-2} \cdot lb$ (Zone 3)

$p_2 := q_h \cdot \left(\left(GC_{p2oh}\right)\right) psf$ $p_2 = -67.63\, ft^{-2} \cdot lb$ (Zone 2 Overhang)

$p_3 := q_h \cdot \left(\left(GC_{p3oh}\right)\right) psf$ $p_3 = -96.62\, ft^{-2} \cdot lb$ (Zone 3 Overhang)

When roof pitch is less than θ=10 degrees, values of GC_p for walls may be reduced by 10%

$p_4 := q_h \cdot \left[\left(GC_{p4out}\right) - \left(GC_{pi}\right)\right] psf$ $p_4 = -30.92\, ft^{-2} \cdot lb$ (Zone 4)

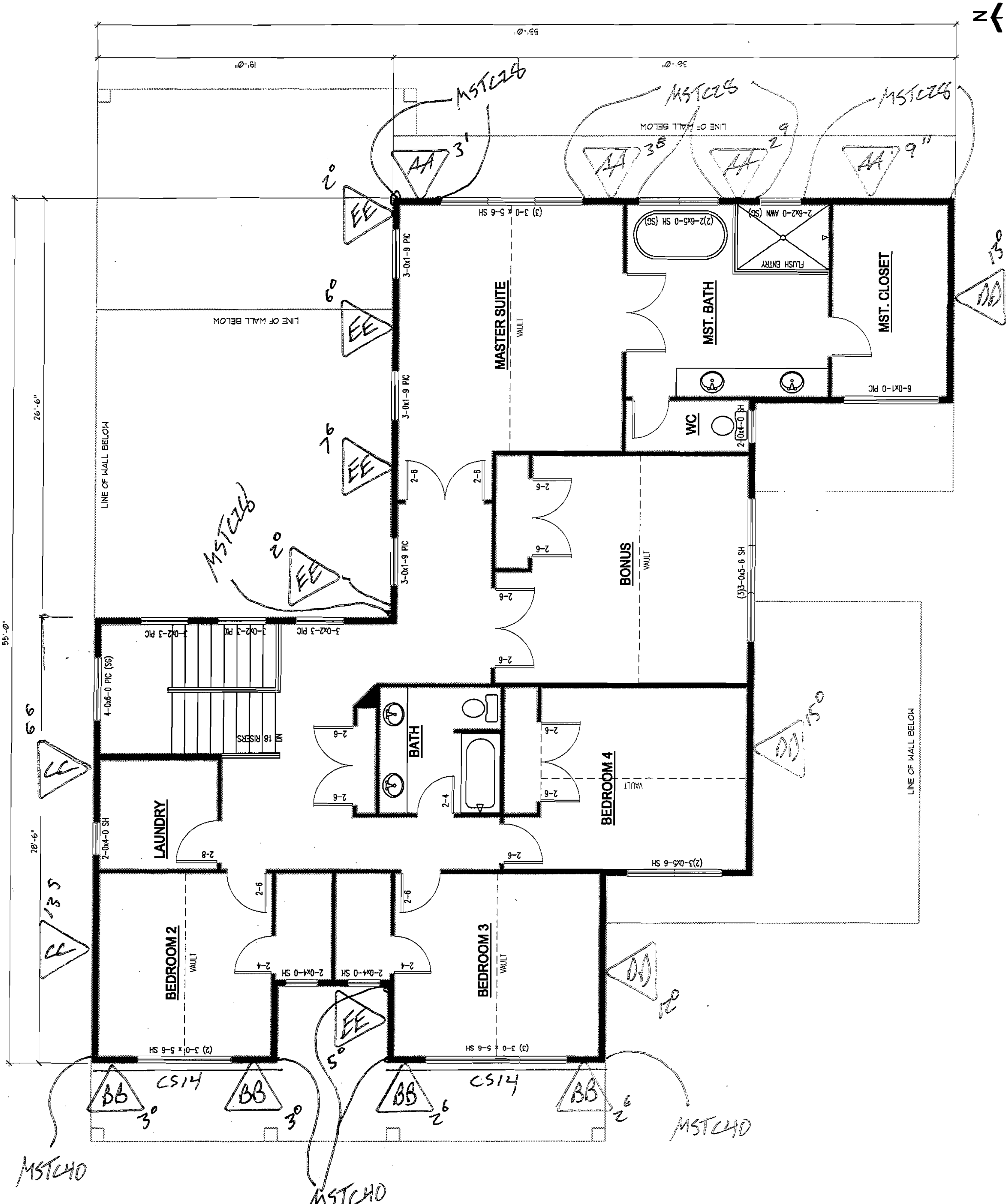
$p_5 := q_h \cdot \left[\left(GC_{p5out}\right) - \left(GC_{pi}\right)\right] psf$ $p_5 = -38.17\, ft^{-2} \cdot lb$ (Zone 5)

Net pressure shall not be less than 16 psf for Components and Cladding (ASCE 7-16 Sec. 30.2.2)

a = 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than 4% of least horizontal dimension or 3ft

$0.1(55ft) = 5.5\, ft$ $0.4 \cdot h = 9.6\, ft$ $0.04(55ft) = 2.2\, ft$

Therefore $a := 5.5ft$



55'-0"
19'-0"
36'-0"
26'-6"
28'-6"
LINE OF WALL BELOW
MASTER SUITE
VAULT
MST. BATH
MST. CLOSET
FLUSH ENTRY
WC
BONUS
BATH
BEDROOM 4
BEDROOM 2
BEDROOM 3
LAUNDRY
18 RISERS
(3) 3-0 x 5-6 SH
(2)2-6x5-0 SH (SG)
2-6x2-0 AWN (SG)
3-0x1-9 PIC
6-0x1-0 PIC
2-0x4-0 SH
(3)3-0x5-6 SH
(2)3-0x5-6 SH
3-0x2-3 PIC
4-0x6-0 PIC (SG)
(2) 3-0 x 5-6 SH
MSTC28
MSTC40
CS14
AA
BB
CC
DD
EE

12

WALL AA:

Story Shear due to Wind: $V_{3W} = 14421.24\,\text{lb}$ Story Shear due to Seismic: $F_1 = 12783.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55 \cdot \text{ft}$ Distance between shear walls: $L_1 := 55 \cdot \text{ft}$

Shear Wall Length:

$Laa_w := (3.083 + 3.667 + 2.75 + 9.917)\text{ft} = 19.42\,\text{ft}$

$Laa_s := \left[3.083\left(\frac{6.17}{9}\right) + 3.667\left(\frac{7.33}{9}\right) + 2.75\left(\frac{5.5}{9}\right) + 9.917\right]\text{ft} = 16.7\,\text{ft}$

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: $vaa := \frac{\frac{0.6V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Laa_w}$

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

$vaa = 222.81\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{vaa}{C_o} = 222.81\,\text{ft}^{-1} \cdot \text{lb}$

$E_{aa} = 267.96\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{E_{aa}}{C_o} = 267.96\,\text{ft}^{-1} \cdot \text{lb}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 495 plf
Seismic Capacity = 353 plf

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

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

$DLRaa := \frac{W_{aa} \cdot L_{aa}}{2}$ $DLRaa = 165\,\text{lb}$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_w = 2005.32\,\text{lb}$

$CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_s = 2411.6\,\text{lb}$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 1906.32\,\text{lb}$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 2338.81\,\text{lb}$

Simpson MSTC28 to flush beam

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_p := \frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 0.73\,\text{ft}$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 0.61\,\text{ft}$

16d @ 6" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{(Z_B \cdot C_o)}{vaa} = 6.18\,\text{ft}$ $\frac{(Z_B \cdot C_o)}{E_{aa}} = 5.14\,\text{ft}$

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

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WALL BB:

Story Shear due to Wind: $V_{3W} = 14421.24\,lb$

Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$

Distance between shear walls: $L_1 := 55 \cdot ft$

Shear Wall Length: $Lbb_w := (2 \cdot 3 + 2 \cdot 2.5)ft = 11\,ft$

$$Lbb_s := \left[2 \cdot 3 + 2 \cdot 2.5\left(\frac{5}{5.5}\right)\right]ft = 10.55\,ft$$

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: $$vbb := \frac{\frac{0.6 V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Lbb_w}$$

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

$vbb = 393.31\,ft^{-1} \cdot lb$ $\quad \frac{vbb}{C_o} = 393.31\,ft^{-1} \cdot lb$

$E_{bb} = 424.28\,ft^{-1} \cdot lb$ $\quad \frac{E_{bb}}{C_o} = 424.28\,ft^{-1} \cdot lb$

P1-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 638 plf
Seismic Capacity = 456 plf

Dead Load Resisting Overturning: $L_{bb} := 12 \cdot ft$ $\quad$ Plate Height: $Pt := 9 \cdot ft$

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

$$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2} \qquad DLRbb = 720\,lb$$

Chord Force:

$$CFbb_w := \frac{vbb \cdot 6ft \cdot Pt}{C_o \cdot L_{bb}} \qquad CFbb_w = 1769.88\,lb$$

$$CFbb_s := \frac{E_{bb} \cdot 6ft \cdot Pt}{C_o \cdot L_{bb}} \qquad CFbb_s = 1909.26\,lb$$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 1337.88\,lb$

$HDFbb_s := CFbb_s - (0.6 - 0.14 S_{DS}) \cdot DLRbb = 1591.64\,lb$

Simpson MSTC40

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 0.41\,ft \qquad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.38\,ft$$

16d @ 4" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$$As := \frac{(Z_B \cdot C_o)}{vbb} = 3.5\,ft \qquad \frac{(Z_B \cdot C_o)}{E_{bb}} = 3.24\,ft$$

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 13566.21\,lb$ Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$ Distance between shear walls: $L_1 := 19 \cdot ft$

Shear Wall Length: $Lcc_w := (13.417 + 6.5)ft = 19.92\,ft$ $Lcc_s := (13.417 + 6.5)ft = 19.92\,ft$

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

$vcc = 70.59\,ft^{-1} \cdot lb$ $\frac{vcc}{C_o} = 70.59\,ft^{-1} \cdot lb$ $E_{cc} = 77.6\,ft^{-1} \cdot lb$ $\frac{E_{cc}}{C_o} = 77.6\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 6.5 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{cc} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRcc := \frac{W_{cc} \cdot L_{cc}}{2}$ $DLRcc = 390\,lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_w = 635.32\,lb$ $CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_s = 698.44\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 401.32\,lb$ $HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 526.39\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_N := \frac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 2.31\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 2.1\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{(Z_B \cdot C_o)}{vcc} = 19.49\,ft$ $\frac{(Z_B \cdot C_o)}{E_{cc}} = 17.73\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 13566.21\,lb$

Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$

Distance between shear walls: $L_1 := 36\cdot ft$

Shear Wall Length: $Ldd_w := (13 + 15 + 12)ft = 40\,ft$

$Ldd_s := (13 + 15 + 12)ft = 40\,ft$

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: $vdd := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1}{2}}{Ldd_w}$

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

$vdd = 66.6\,ft^{-1}\cdot lb$ $\frac{vdd}{C_o} = 66.6\,ft^{-1}\cdot lb$

$E_{dd} = 73.21\,ft^{-1}\cdot lb$ $\frac{E_{dd}}{C_o} = 73.21\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 12\cdot ft$ Plate Height: $Pt := 9\cdot ft$

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

$DLRdd := \frac{W_{dd}\cdot L_{dd}}{2}$ $DLRdd = 720\,lb$

Chord Force:

$CFdd_w := \frac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_w = 599.38\,lb$

$CFdd_s := \frac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_s = 658.93\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 167.38\,lb$

$HDFdd_s := CFdd_s - \left(0.6 - 0.14S_{DS}\right)DLRdd = 341.31\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_n := \frac{\left(C_D\cdot Z_N\cdot C_o\right)}{vdd} = 2.45\,ft$ $\frac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{dd}} = 2.23\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{\left(Z_B\cdot C_o\right)}{vdd} = 20.66\,ft$ $\frac{\left(Z_B\cdot C_o\right)}{E_{dd}} = 18.79\,ft$

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

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WALL EE:

Story Shear due to Wind: $V_{1W} = 13566.21\,\text{lb}$ Story Shear due to Seismic: $F_1 = 12783.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55\cdot\text{ft}$ Distance between shear walls: $L_1 := 19\cdot\text{ft}$ $L_2 := 36\text{ft}$

Shear Wall Length: $Lee_w := (2\cdot2 + 7.5 + 6 + 5)\text{ft} = 22.5\,\text{ft}$ $Lee_s := (2\cdot2 + 7.5 + 6 + 5)\text{ft} = 22.5\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{17.5\cdot\text{ft}}{26.5\cdot\text{ft}}\right)\cdot100$ $\% = 66.04$ Max Opening Height = 1ft-6in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vee := \frac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lee_w}$ Seismic Force: $\rho := 1.0$ $E_{ee} := \frac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lee_s}$

$vee = 180.88\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{vee}{C_o} = 180.88\,\text{ft}^{-1}\cdot\text{lb}$ $E_{ee} = 198.85\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_{ee}}{C_o} = 198.85\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_{ee} := 5\cdot\text{ft}$ Plate Height: $Pt := 9\cdot\text{ft}$

$W_{ee} := (15\cdot\text{psf})\cdot8\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot0\text{ft}$ $DLRee := \frac{W_{ee}\cdot L_{ee}}{2}$ $DLRee = 525\,\text{lb}$

Chord Force:

$CFee_w := \frac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_w = 1627.95\,\text{lb}$ $CFee_s := \frac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_s = 1789.69\,\text{lb}$

Holdown Force:

$HDFee_w := CFee_w - 0.6\cdot DLRee = 1312.95\,\text{lb}$ $HDFee_s := CFee_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRee = 1558.09\,\text{lb}$

Simpson MSTC40 to wall or MSTC28 to flush beam

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_R := \frac{\left(C_D\cdot Z_N\cdot C_o\right)}{vee} = 0.9\,\text{ft}$ $\frac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{ee}} = 0.82\,\text{ft}$

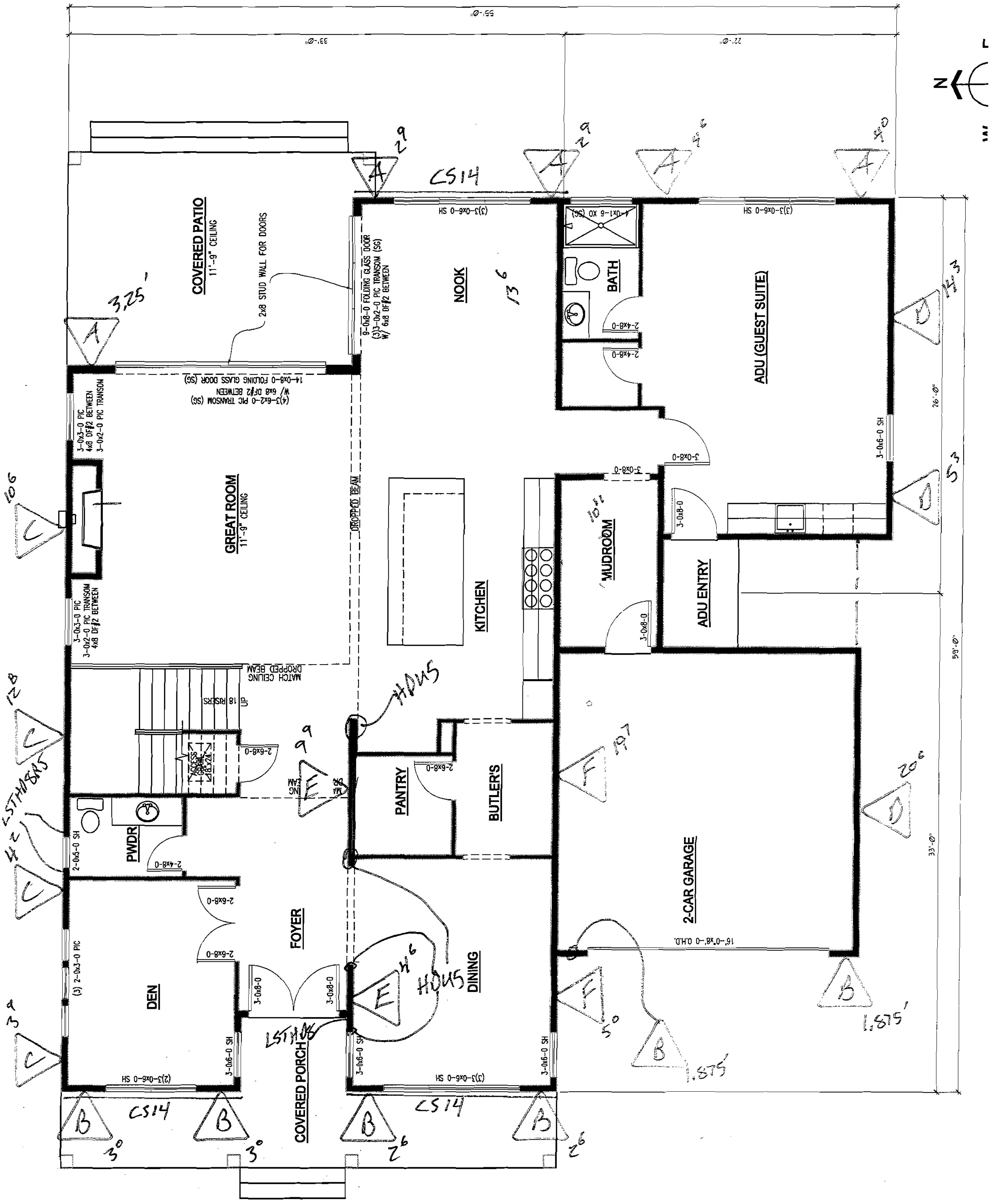
16d @ 8" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$A_s := \frac{\left(Z_B\cdot C_o\right)}{vee} = 7.61\,\text{ft}$ $\frac{\left(Z_B\cdot C_o\right)}{E_{ee}} = 6.92\,\text{ft}$

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


55'-0"
33'-0"
22'-0"
COVERED PATIO
11'-9" CEILING
2x8 STUD WALL FOR DOORS
9-0x8-0 FOLDING GLASS DOOR
(3)3-0x2-0 PIC TRANSOM (SG)
W/ 6x8 DF#2 BETWEEN
14-0x8-0 FOLDING GLASS DOOR (SG)
W/ 6x8 DF#2 BETWEEN
(4)3-6x2-0 PIC TRANSOM (SG)
NOOK
(3)3-0x6-0 SH
4-0x1-6 XO (SG)
BATH
2-4x8-0
ADU (GUEST SUITE)
3-0x6-0 SH
3-0x8-0
GREAT ROOM
11'-9" CEILING
DROPPED BEAM
3-0x3-0 PIC
4x8 DF#2 BETWEEN
3-0x2-0 PIC TRANSOM
KITCHEN
MUDROOM
ADU ENTRY
26'-0"
59'-0"
33'-0"
MATCH CEILING
DROPPED BEAM
UP
18 RISERS
2-6x8-0
PANTRY
BUTLER'S
PWDR
2-0x5-0 SH
2-4x8-0
FOYER
DEN
(3) 2-0x3-0 PIC
3-0x8-0
DINING
2-CAR GARAGE
16'-0"x8'-0" O.H.D.
COVERED PORCH
(2)3-0x6-0 SH
(3)3-0x6-0 SH
CS14
CS14
CS14
HDU5
HDU5
A
B
C
D
E
F
3.25'
1.875'
1.875'

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WALL A:

Story Shear due to Wind: $V_{4W} = 16007.84\,\mathrm{lb}$ Story Shear due to Seismic: $F_2 = 10031.78\,\mathrm{lb}$

Bldg Width in direction of Load: $L_t := 59\cdot\mathrm{ft}$ Distance between shear walls: $L_1 := 59\cdot\mathrm{ft}$

Shear Wall Length:

$La_w := (3.25 + 2\cdot 2.75 + 4.5 + 4)\mathrm{ft} = 17.25\,\mathrm{ft}$

$La_s := \left[3.25\left(\frac{6.5}{10}\right) + 2\cdot 2.75\left(\frac{5.5}{6}\right) + 4.5\left(\frac{9}{10}\right) + 4\left(\frac{8}{10}\right)\right]\mathrm{ft} = 14.4\,\mathrm{ft}$

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

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

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

$va = 529.2\,\mathrm{ft}^{-1}\cdot\mathrm{lb} \qquad \frac{va}{C_o} = 529.2\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$

$E_a = 554.38\,\mathrm{ft}^{-1}\cdot\mathrm{lb} \qquad \frac{E_a}{C_o} = 554.38\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 833 plf
Seismic Capacity = 595 plf

Dead Load Resisting Overturning: $L_a := 3.25\cdot\mathrm{ft}$ Plate Height: $Pt := 10\cdot\mathrm{ft}$

$W_a := (15\cdot\mathrm{psf})\cdot 2\cdot\mathrm{ft} + (10\cdot\mathrm{psf})\cdot Pt + (10\mathrm{psf})\cdot 0\mathrm{ft}$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 211.25\,\mathrm{lb}$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 5292.01\,\mathrm{lb}$ $CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 5543.78\,\mathrm{lb}$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 5165.26\,\mathrm{lb}$ $HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 5450.59\,\mathrm{lb}$

Simpson HDU5 at DF post w/ SB5/8x24 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.31\,\mathrm{ft} \qquad \frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 0.29\,\mathrm{ft}$

16d @ 3" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$A_s := \frac{(Z_B\cdot C_o)}{va} = 2.6\,\mathrm{ft} \qquad \frac{(Z_B\cdot C_o)}{E_a} = 2.48\,\mathrm{ft}$

5/8" A.B. @ 30" o.c.

WALL B:

Story Shear due to Wind: $V_{4W} = 16007.84\,lb$

Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 59\cdot ft$

Distance between shear walls: $L_1 := 59\cdot ft$

Shear Wall Length: $Lb_w := (2\cdot 3 + 2\cdot 2.5 + 2\cdot 1.875)ft = 14.75\,ft$

$Lb_s := \left[2\cdot 3 + 2\cdot 2.5\left(\frac{5}{6}\right) + 2\cdot 1.875\right]ft = 13.92\,ft$

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: $vb := \dfrac{vbb\cdot Lbb_w + \left(\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lb_w}$

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

$vb = 618.9\,ft^{-1}\cdot lb$ $\quad \dfrac{vb}{C_o} = 618.9\,ft^{-1}\cdot lb$

$E_b = 573.8\,ft^{-1}\cdot lb$ $\quad \dfrac{E_b}{C_o} = 573.8\,ft^{-1}\cdot lb$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 833 plf
Seismic Capacity = 595 plf

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

Restraint Panel Height = 9ft Maximum

Restraint Panel Width = 1ft-10-1/2 in Minimum

Allowable Shear per Panel = 1187 lbs Seismic & 1661 lbs Wind

Shear per Panel: $V_s := \left(1.875ft\cdot E_b\right) = 1075.87\,lb$ O.K.

$V_w := (1.875ft\cdot vb) = 1160.43\,lb$ O.K.

Dead Load Resisting Overturning: $L_b := 12\cdot ft$ $\quad$ Plate Height: $Pt := 10\cdot ft$

$W_b := (15\cdot psf)\cdot 2.5\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$

$DLRb := \dfrac{W_b\cdot L_b}{2}$ $\quad DLRb = 885\,lb$

Chord Force:

$CFb_w := \dfrac{vb\cdot 6ft\cdot Pt}{C_o\cdot L_b}$ $\quad CFb_w = 3094.48\,lb$

$CFb_w + CFbb_w = 4864.36\,lb$

$CFb_s := \dfrac{E_b\cdot 6ft\cdot Pt}{C_o\cdot L_b}$ $\quad CFb_s = 2868.99\,lb$

$CFb_s + CFbb_s = 4778.25\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 2563.48\,lb$

$HDFb_w + HDFbb_w = 3901.36\,lb$

$HDFb_s := CFb_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRb = 2478.57\,lb$

$HDFb_s + HDFbb_s = 4070.21\,lb$

Simpson HDU5 w/ SB5/8x24 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_R := \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{vb} = 0.26\,ft$ $\quad \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_b} = 0.28\,ft$

16d @ 3" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$A_S := \dfrac{\left(Z_B\cdot C_o\right)}{vb} = 2.22\,ft$ $\quad \dfrac{\left(Z_B\cdot C_o\right)}{E_b} = 2.4\,ft$

5/8" A.B. @ 24" o.c.

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PROJECT : **42xx 89th AVE SE**

WALL C:

Story Shear due to Wind: $V_{2W} = 14653.33\,lb$

Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$

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

Shear Wall Length: $Lc_w := (3.75 + 4.17 + 12.67 + 10.5)ft = 31.09\,ft$

$$Lc_s := \left[3.75\left(\frac{7.5}{10}\right) + 4.17\left(\frac{8.33}{10}\right) + 12.67 + 10.5\right]ft = 29.46\,ft$$

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: $$vc := \frac{vcc\cdot Lcc_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lc_w}$$

Seismic Force: $\rho := 1.0$ $$E_c := \frac{E_{cc}\cdot Lcc_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Lc_s}$$

$vc = 94.07\,ft^{-1}\cdot lb$ $\quad \frac{vc}{C_o} = 94.07\,ft^{-1}\cdot lb$ $\quad E_c = 93.65\,ft^{-1}\cdot lb$ $\quad \frac{E_c}{C_o} = 93.65\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_c := 3.75\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_c := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 6ft$ $\quad DLRc := \frac{W_c\cdot L_c}{2}$ $\quad DLRc = 300\,lb$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_w = 940.68\,lb$ $\quad CFc_w + CFcc_w = 1575.99\,lb$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_s = 936.5\,lb$ $\quad CFc_s + CFcc_s = 1634.94\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 760.68\,lb$ $\quad HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 804.16\,lb$

No Holdown Required

$HDFc_w + HDFcc_w = 1161.99\,lb$ $\quad HDFc_s + HDFcc_s = 1330.55\,lb$

Simpson LSTHD8RJ

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_N := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 1.73\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 1.74\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{(Z_B\cdot C_o)}{vc} = 14.63\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_c} = 14.69\,ft$

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

WALL D:

Story Shear due to Wind: $V_{2W} = 14653.33\,\text{lb}$

Story Shear due to Seismic: $F_2 = 10031.78\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55 \cdot \text{ft}$

Distance between shear walls: $L_1 := 22 \cdot \text{ft}$

Shear Wall Length: $Ld_w := (5.25 + 14.25 + 20.5)\text{ft} = 40\,\text{ft}$

$Ld_s := (5.25 + 14.25 + 20.5)\text{ft} = 40\,\text{ft}$

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: $vd := \dfrac{vdd \cdot Ldd_w + \left(\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1}{2}\right)}{Ld_w}$

Seismic Force: $\rho := 1.0$ $E_d := \dfrac{E_{dd} \cdot Ldd_s + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Ld_s}$

$vd = 110.56\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{vd}{C_o} = 110.56\,\text{ft}^{-1} \cdot \text{lb}$

$E_d = 108.33\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{E_d}{C_o} = 108.33\,\text{ft}^{-1} \cdot \text{lb}$

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

Dead Load Resisting Overturning: $L_d := 5.25 \cdot \text{ft}$ Plate Height: $Pt := 10 \cdot \text{ft}$

$W_d := (15 \cdot \text{psf}) \cdot 0 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 8\text{ft}$

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 472.5\,\text{lb}$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 1105.58\,\text{lb}$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 1083.26\,\text{lb}$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 822.08\,\text{lb}$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 874.82\,\text{lb}$

No Holdown Required

Dead Load Resisting Overturning: $L_d := 14.25 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_d := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 8ft$

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 1282.5\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 1105.58\,lb$

$CFd_w + CFdd_w = 1704.96\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 1083.26\,lb$

$CFd_s + CFdd_s = 1742.19\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 336.08\,lb$

$HDFd_w + HDFdd_w = 503.46\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 517.49\,lb$

$HDFd_s + HDFdd_s = 858.8\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 1.48\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 1.51\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{(Z_B \cdot C_o)}{vd} = 12.45\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 12.7\,ft$

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

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WALL E:

Story Shear due to Wind: $V_{2W} = 14653.33\,\text{lb}$ Story Shear due to Seismic: $F_2 = 10031.78\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55\cdot\text{ft}$ Distance between shear walls: $L_1 := 19\cdot\text{ft}$ $L_2 := 14\text{ft}$

Shear Wall Length: $Le_w := (9.75 + 4.5)\text{ft} = 14.25\,\text{ft}$ $Le_s := \left[9.75 + 4.5\left(\frac{9}{10}\right)\right]\text{ft} = 13.8\,\text{ft}$

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: $$ve := \frac{vee\cdot Lee_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Le_w}$$

Seismic Force: $\rho := 1.0$ $$E_e := \frac{E_{ee}\cdot Lee_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Le_s}$$

$ve = 470.7\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{ve}{C_o} = 470.7\,\text{ft}^{-1}\cdot\text{lb}$ $E_e = 476.88\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_e}{C_o} = 476.88\,\text{ft}^{-1}\cdot\text{lb}$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 833 plf
Seismic Capacity = 595 plf

Dead Load Resisting Overturning: $L_e := 4.5\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_e := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 10\text{ft}$ $DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 450\,\text{lb}$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 4706.99\,\text{lb}$ $CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 4768.77\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 4436.99\,\text{lb}$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 4570.26\,\text{lb}$

Simpson HDU5 w/ SB5/8x24 or PAB5 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot\text{lb}$ $C_D := 1.6$
$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.35\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.34\,\text{ft}$

16d @ 4" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot\text{lb}$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,\text{lb}$
$As := \frac{(Z_B\cdot C_o)}{ve} = 2.92\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_e} = 2.89\,\text{ft}$

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

WALL F:

Story Shear due to Wind: $V_{2W} = 14653.33\,lb$ Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$ Distance between shear walls: $L_1 := 14 \cdot ft$ $L_2 := 22ft$

Shear Wall Length: $Lf_w := (5 + 19.58)ft = 24.58\,ft$ $Lf_s := (5 + 19.58)ft = 24.58\,ft$

Percent full height sheathing: $\% := \left(\frac{24.58 \cdot ft}{25.08 \cdot ft}\right) \cdot 100$ $\% = 98.01$ Max Opening Height = 10ft-0in, Therefore $C_o := 0.95$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vf := \dfrac{\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf_w}$ Seismic Force: $\rho := 1.0$ $E_f := \dfrac{\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf_s}$

$vf = 117.06\,ft^{-1} \cdot lb$ $\frac{vf}{C_o} = 123.22\,ft^{-1} \cdot lb$ $E_f = 93.5\,ft^{-1} \cdot lb$ $\frac{E_f}{C_o} = 98.42\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_f := 24.58 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 7ft$ $DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 2089.3\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 1232.23\,lb$ $CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 984.19\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = -21.35\,lb$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 62.51\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

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

$B_p := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 1.32\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 1.66\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

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

$As := \frac{(Z_B \cdot C_o)}{vf} = 11.17\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 13.98\,ft$

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

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa\cdot\frac{Laa_w}{36ft} = 120.18\,ft^{-1}\cdot lb \qquad E_{aa}\cdot\frac{Laa_s}{36ft} = 124.28\,ft^{-1}\cdot lb$$

Wall Lines DD:

$$vdd\cdot\frac{Ldd_w}{55ft} = 48.43\,ft^{-1}\cdot lb \qquad E_{dd}\cdot\frac{Ldd_s}{55ft} = 53.25\,ft^{-1}\cdot lb$$

Wall Lines BB:

$$vbb\cdot\frac{Lbb_w}{33ft} = 131.1\,ft^{-1}\cdot lb \qquad E_{bb}\cdot\frac{Lbb_s}{33ft} = 135.58\,ft^{-1}\cdot lb$$

Wall Lines EE:

$$vee\cdot\frac{Lee_w}{55ft} = 74\,ft^{-1}\cdot lb \qquad E_{ee}\cdot\frac{Lee_s}{55ft} = 81.35\,ft^{-1}\cdot lb$$

Wall Lines CC:

$$vcc\cdot\frac{Lcc_w}{28ft} = 50.21\,ft^{-1}\cdot lb \qquad E_{cc}\cdot\frac{Lcc_s}{28ft} = 55.2\,ft^{-1}\cdot lb$$

Wall Lines A:

$$\frac{va\cdot La_w - vaa\cdot Laa_w}{55ft} = 87.32\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La_s - E_{aa}\cdot Laa_s}{55ft} = 63.84\,ft^{-1}\cdot lb$$

Wall Lines B:

$$\frac{vb\cdot Lb_w - vbb\cdot Laa_w}{55ft} = 27.13\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb_s - E_{bb}\cdot Lbb_w}{55ft} = 60.33\,ft^{-1}\cdot lb$$

Wall Lines C:

$$\frac{vc\cdot Lc_w - vcc\cdot Lcc_w}{59ft} = 25.74\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc_s - E_{cc}\cdot Lcc_s}{59ft} = 20.56\,ft^{-1}\cdot lb$$

Wall Lines D:

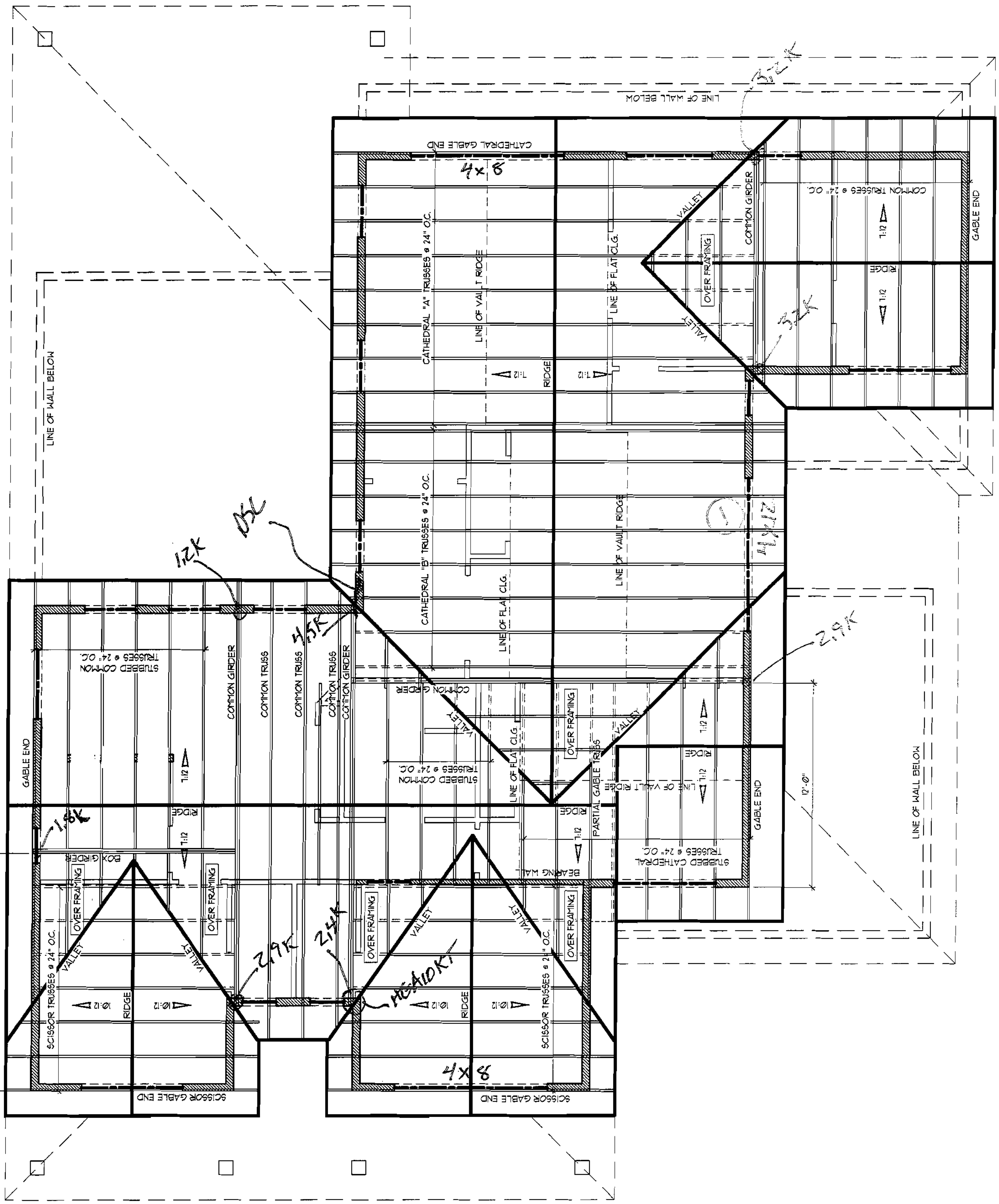
$$\frac{vd\cdot Ld_w - vdd\cdot Ldd_w}{50ft} = 35.17\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld_s - E_{dd}\cdot Ldd_s}{50ft} = 28.09\,ft^{-1}\cdot lb$$

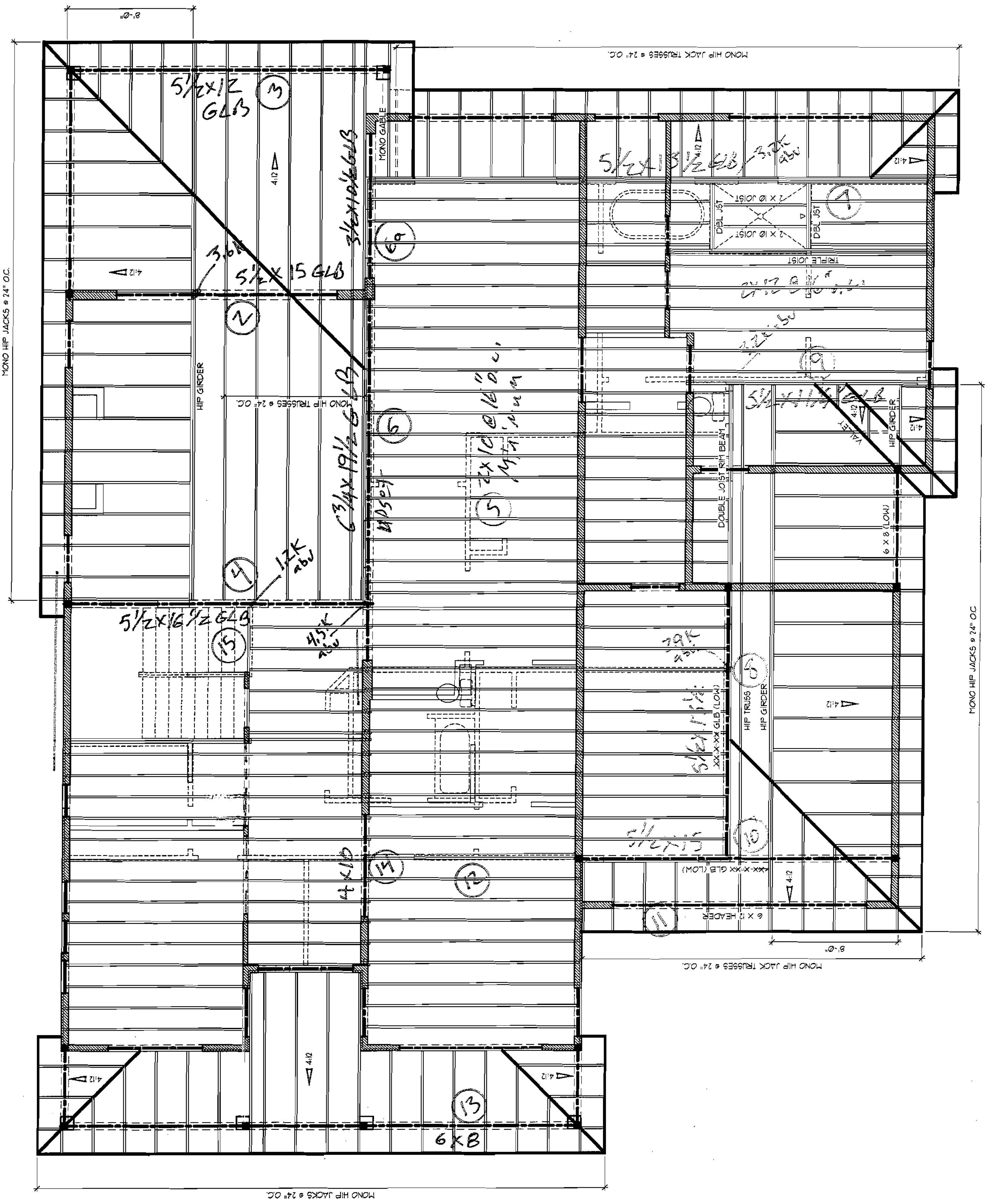
Wall Line E:

$$\frac{ve\cdot Le_w - vee\cdot Ldd_w}{55ft} = -9.6\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le_s - E_{ee}\cdot Ldd_s}{55ft} = -24.97\,ft^{-1}\cdot lb$$

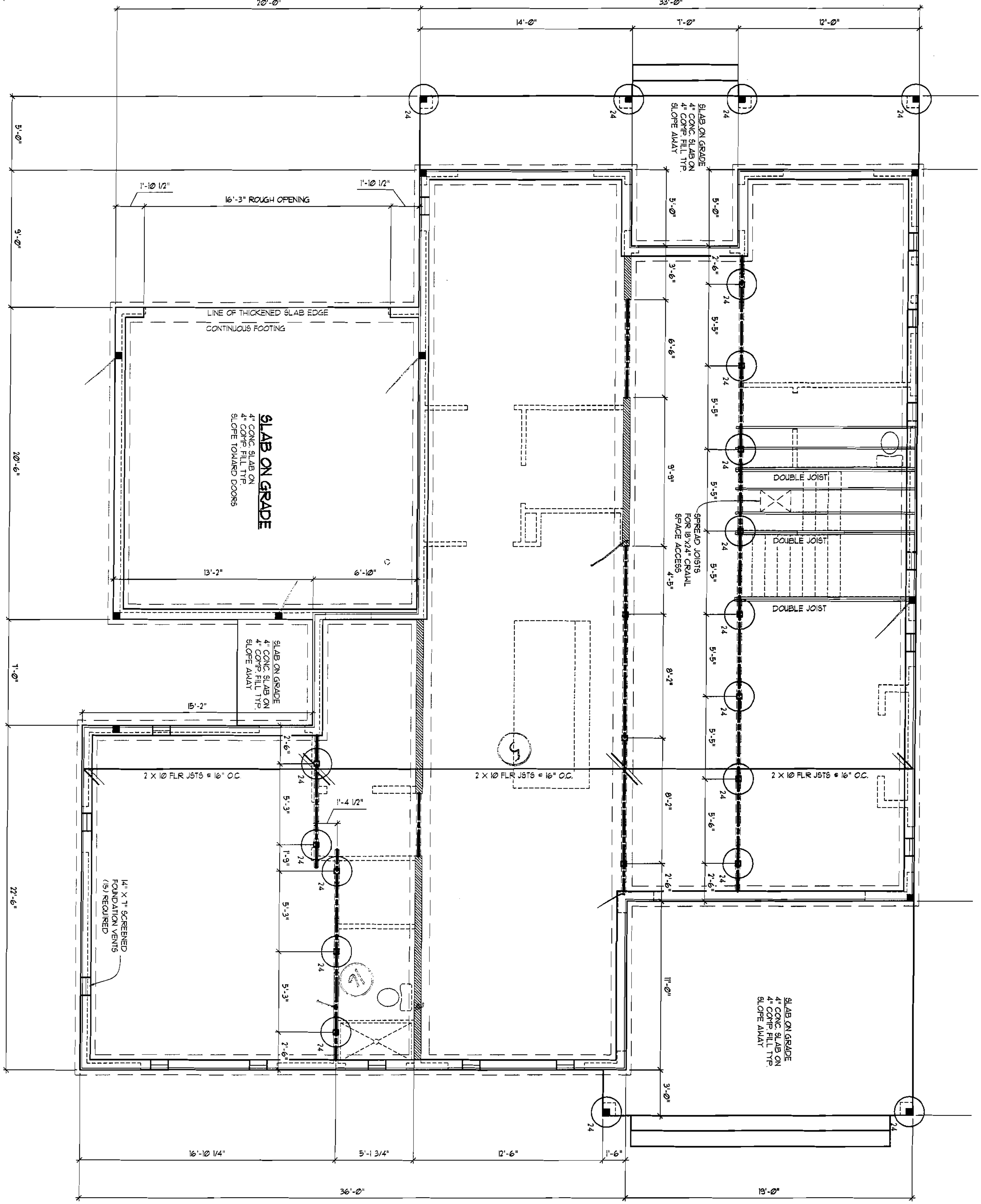
Wall Line F:

$$\frac{vf\cdot Lf_w}{59ft} = 48.77\,ft^{-1}\cdot lb \qquad \frac{E_f\cdot Lf_s}{59ft} = 38.95\,ft^{-1}\cdot lb$$


LINE OF WALL BELOW
CATHEDRAL GABLE END
4×8
COMMON TRUSSES @ 24" O.C.
GABLE END
VALLEY
COMMON GIRDER
OVER FRAMING
RIDGE
1:12
3.2K
CATHEDRAL "A" TRUSSES @ 24" O.C.
LINE OF VAULT RIDGE
LINE OF FLAT CLG.
CATHEDRAL "B" TRUSSES @ 24" O.C.
12K
DSC
4.5K
STUBBED COMMON TRUSSES @ 24" O.C.
COMMON GIRDER
COMMON TRUSS
COMMON GIRDER
STUBBED CATHEDRAL TRUSSES @ 24" O.C.
PARTIAL GABLE TRUSS
BEARING WALL
2.9K
12'-0"
14'-0"
1.8K
BOX GIRDER
SCISSOR TRUSSES @ 24" O.C.
10:12
2.9K
2.4K
HGA10KT
SCISSOR GABLE END
4×8


8'-0"
MONO HIP JACK TRUSSES @ 24" O.C.
MONO HIP JACKS @ 24" O.C.
MONO GABLE
5 1/2 x 12 GLB
5 1/2 x 15 GLB
HIP GIRDER
MONO HIP TRUSSES @ 24" O.C.
2 X 10 JOIST
DBL JST
TRIPLE JOIST
DOUBLE JOIST RIM BEAM
VALLEY
6 x 8 (LOW)
HIP TRUSS
XX-X-XX GLB (LOW)
6 X 12 HEADER
4:12
4 x 10
6 x 8

29



20'-0"
33'-0"
14'-0"
7'-0"
12'-0"
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
1'-10 1/2"
16'-3" ROUGH OPENING
LINE OF THICKENED SLAB EDGE
CONTINUOUS FOOTING
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE TOWARD DOORS
13'-2"
6'-10"
DOUBLE JOIST
SPREAD JOISTS
FOR 18"x24" CRAWL
SPACE ACCESS
15'-2"
2 X 10 FLR JSTS @ 16" O.C.
1'-4 1/2"
14" X 7" SCREENED
FOUNDATION VENTS
(15) REQUIRED
5'-0"
9'-0"
20'-6"
7'-0"
22'-6"
11'-0"
3'-0"
16'-10 1/4"
5'-1 3/4"
12'-6"
1'-6"
36'-0"
19'-0"

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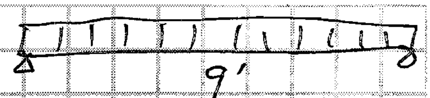
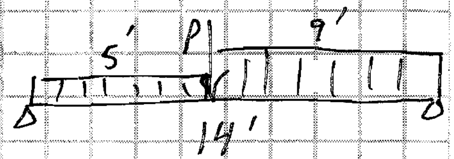
① $w_D = 15psf(27'/2) = 202.5plf$
$w_S = 25psf(27'/2) = 337.5plf$



4x12 DF #2
(4x8 @ 6' span)

② $w_{D1} = 15psf(2') = 30plf$
$w_{S1} = 25psf(2') = 50plf$

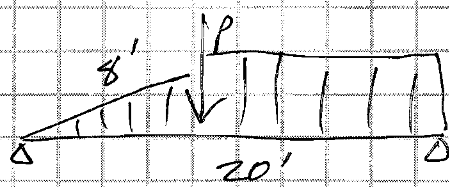


5½x15 GLB

$P = 1445^{\#}DL + 2125^{\#}SL$ from Girder

$w_{D2} = 15psf(33'/2)(1.25) = 310plf$
$w_{S2} = 25psf(33'/2)(1.25) = 516plf$

③ $w_{D1} = 15psf(8'/2) = 60plf$
$w_{S1} = 25psf(8'/2) = 100plf$

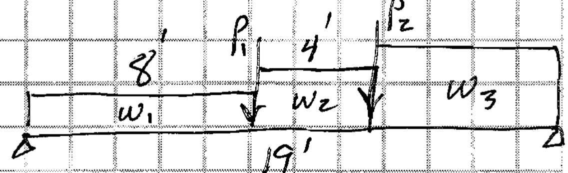


5½x12 GLB

$P = 420^{\#}DL + 700^{\#}SL$ from Girder

$w_{D2} = 15psf(17'/2) = 127.5plf$
$w_{S2} = 25psf(17'/2) = 212.5plf$

④ $w_{D1} = 15psf(17'/2 + 1') + 12psf(9') = 251plf$
$w_{S1} = 25psf(17'/2 + 1') = 238plf$



5½x16½ GLB

$P_1 = 570^{\#}DL + 950^{\#}SL$ from Girder

$w_{D2} = 15psf(17'/2 + 19'/2) + 12psf(9') = 378plf$
$w_{S2} = 25psf(17'/2 + 19'/2) = 450plf$

$P_2 = 696^{\#}DL + 560^{\#}LL + 715^{\#}SL$ from Girder + header

$w_{D3} = 15psf(27'/2 + 19'/2 + 1') + 12psf(9') = 468plf$
$w_{L3} = 40psf(1') = 40plf$
$w_{S3} = 25psf(27'/2 + 19'/2) = 575plf$

FOR 42xx 89th Ave SE
JOB

DATE 2-12-21
BY MM

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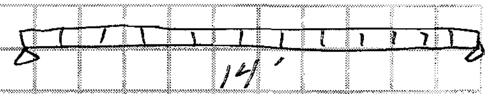
myengineer@centurytel.net

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



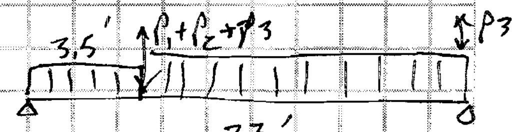
2x10 @ 16" o.c.
(2x12 @ 16.5' span

(6) $w_{D1} = 15psf(21'/2) = 158plf$
$w_{L1} = 40psf(21'/2) = 420plf$

$P_1 = 4575^{\#}DL + 585^{\#}LL + 5485^{\#}SL$ from (4)

$P_2 = 1820^{\#}DL + 2680^{\#}SL$ from Girder

$w_{D2} = 15psf(27'/2 + 14'/2) + 12psf(9') = 416plf$
$w_{L2} = 40psf(14'/2) = 280plf$
$w_{S2} = 25psf(27'/2) = 338plf$
$P_3 = \pm 1630^{\#}WL \pm 1790^{\#}EL$ w/ $\Omega = 3.0$



6 3/4 x 19 1/2 GLB

(3 1/2 x 10 1/2 GLB @ 6a)

(7) $w_{D1} = 15psf(4'/2 + 2' + 1') + 12psf(9') = 183plf$
$w_{L1} = 40psf(1') = 40plf$
$w_{S1} = 25psf(4'/2 + 2') = 100plf$

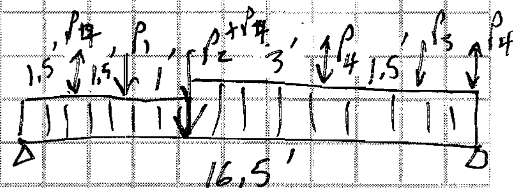
$P_1 = 135^{\#}DL + 360^{\#}LL$ from header

$P_2 = 1295^{\#}DL + 1905^{\#}SL$ from Girder

$w_{D2} = 15psf(4'/2 + 17'/2 + 1') + 12psf(9') = 261plf$
$w_{L2} = 40plf$
$w_{S2} = 25psf(4'/2 + 17'/2) = 263plf$

$P_3 = 210^{\#}DL + 560^{\#}LL$ from header

$P_4 = \pm 2010^{\#}WL \pm 2420^{\#}EL$ w/ $\Omega = 3.0$



5 1/2 x 13 1/2 GLB

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Fax (253) 858-3249
myengineer@centurytel.net

(8) $w_{D1} = 15psf(2' + 9'/2 + 1') + 12psf(9') = 221plf$
$w_{L1} = 40psf(9'/2) = 180plf$
$w_{S1} = 25psf(2' + 1') = 75plf$

$P_1 = 1175\#DL + 1725\#SL$ from Girder

$w_{D2} = 15psf(27'/2 + 9'/2) + 12psf(9') = 378plf$
$w_{L2} = 40psf(9'/2) = 180plf$
$w_{S2} = 25psf(27'/2) = 338plf$

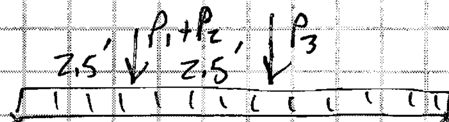
17'
5½x13½ GLB

((2)2x12 @ 7' span)

(9) $w_D = 15psf(1' + 17'/2) + 12psf(9') = 251plf$
$w_L = 40plf$
$w_S = 25psf(17'/2) = 213plf$
$P_1 = 1295\#DL + 1905\#SL$ from Girder abv

$P_2 = 945\#DL + 450\#LL + 845\#SL$ from Rim Beam

$P_3 = 170\#DL + 250\#SL$ from hip Girder



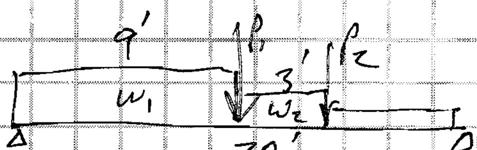
15'
5½x11¼ GLB
Minimum

(10) $w_{D1} = 15psf(18'/2 + 3'/2) + 12psf(9') = 266plf$
$w_{S1} = 25psf(18'/2 + 3'/2) = 263plf$

$P_1 = 2350\#DL + 1550\#LL + 1350\#SL$ from (8)

$w_{D2} = 15psf(20'/2)(1.25) = 188plf$
$w_{S2} = 25psf(20'/2)(1.25) = 313plf$

$P_2 = 850\#DL + 1250\#SL$ from Hip Girder



20'
5½x15 GLB

FOR 42xx 89th Ave SE
JOB
DATE 2-12-21
BY [signature]

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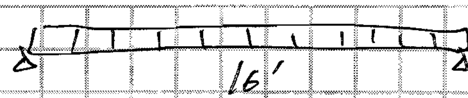
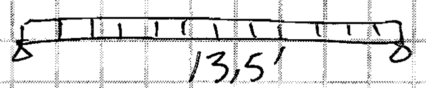
(11) $w_D = 15psf(6'/2) = 45plf$
$w_S = 25psf(6'/2) = 75plf$



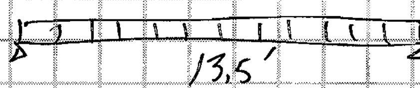
6x8 Minimum

(12) $w_D = 15psf(14'/2 + 1.33') = 125plf$
$w_L = 40psf(1.33') = 53.3plf$
$w_S = 25psf(14'/2) = 175plf$



(3) 2x12

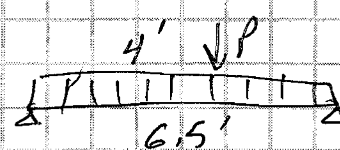
(13) $w_D = 15psf(8'/2) = 60plf$
$w_S = 25psf(8'/2) = 100plf$



6x8

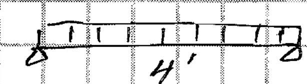
(14) $w_D = 15psf(21'/2) = 158plf$
$w_L = 40psf(21'/2) = 420plf$

$P = 845^{\#}DL + 1180^{\#}SL$ from (12)



4x10

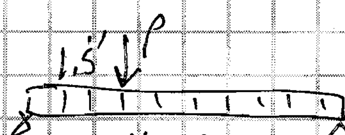
(15) $w_D = 15psf(14'/2) = 105plf$
$w_L = 40psf(14'/2) = 280plf$



2x8 Minimum

(16) $w_D = 15psf(22'/2 + 22'/2) = 330plf$
$w_L = 30psf(22'/2) + 40psf(22'/2) = 770plf$

$P = 3200^{\#}DL + 1000^{\#}LL + 3100^{\#}SL$ from (9)



4x10
2.75' span Max

(5'8" span w/o Pt. Ld.)

FOR 42xx 89th Ave SE
JOB

DATE 2-12-21
BY MM

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

Wood Beam

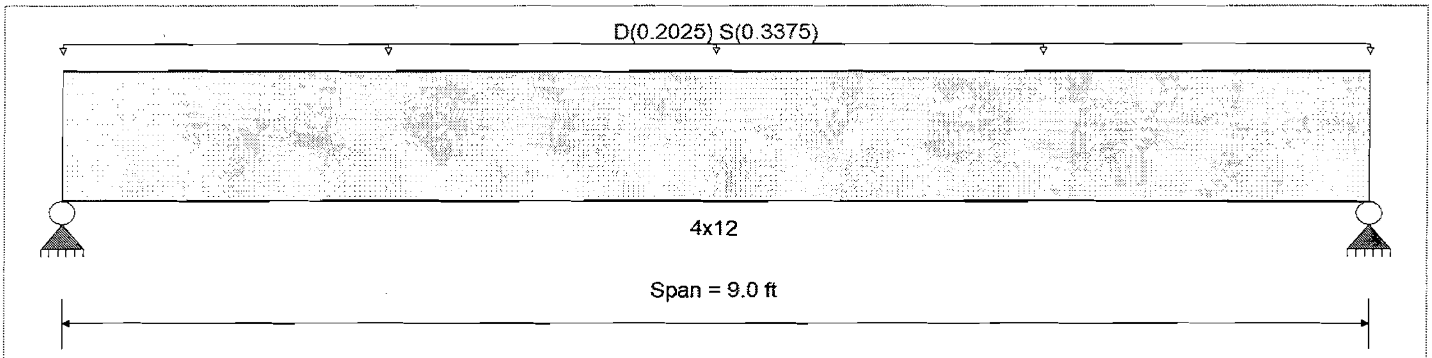
DESCRIPTION: 1. Header

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 +	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 = 0.2025, S = 0.3375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.781**: 1	Maximum Shear Stress Ratio	=	**0.356** : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
	=	888.69 psi		=	73.65 psi
	=	1,138.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.075 in	Ratio =	1431 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.121 in	Ratio =	894 >=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	2.430	2.430
Overall MINimum	1.519	1.519
D Only	0.911	0.911
+D+L	0.911	0.911
+D+S	2.430	2.430
+D+0.750L	0.911	0.911
+D+0.750L+0.750S	2.050	2.050
+0.60D	0.547	0.547
S Only	1.519	1.519

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 1a. Header

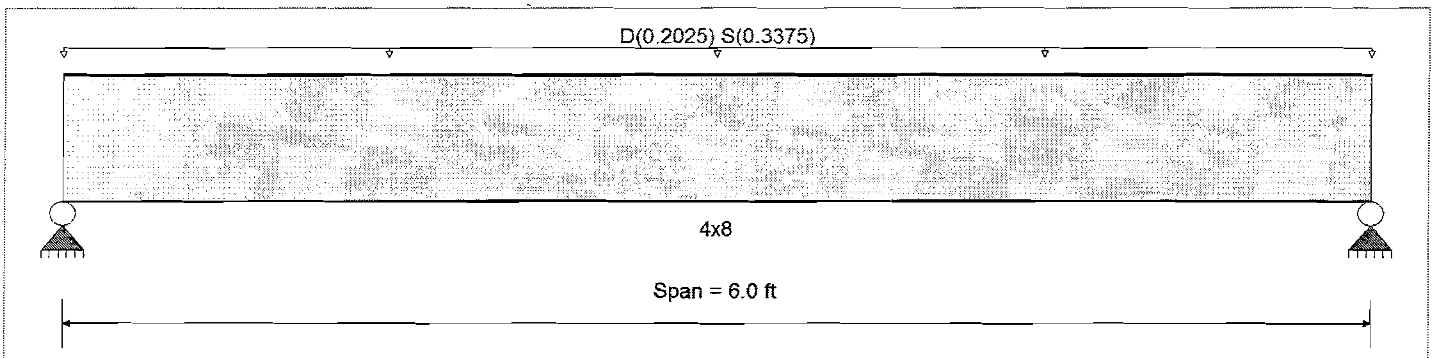
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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 = 0.2025, S = 0.3375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.707: 1**	Maximum Shear Stress Ratio	=	**0.371 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	951.03 psi		=	76.89 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 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	0.056 in	Ratio =	1293 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.089 in	Ratio =	808 >=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	1.620	1.620
Overall MINimum	1.013	1.013
D Only	0.608	0.608
+D+L	0.608	0.608
+D+S	1.620	1.620
+D+0.750L	0.608	0.608
+D+0.750L+0.750S	1.367	1.367
+0.60D	0.365	0.365
S Only	1.013	1.013

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Wood Beam

DESCRIPTION: 2. Header at Folding Door

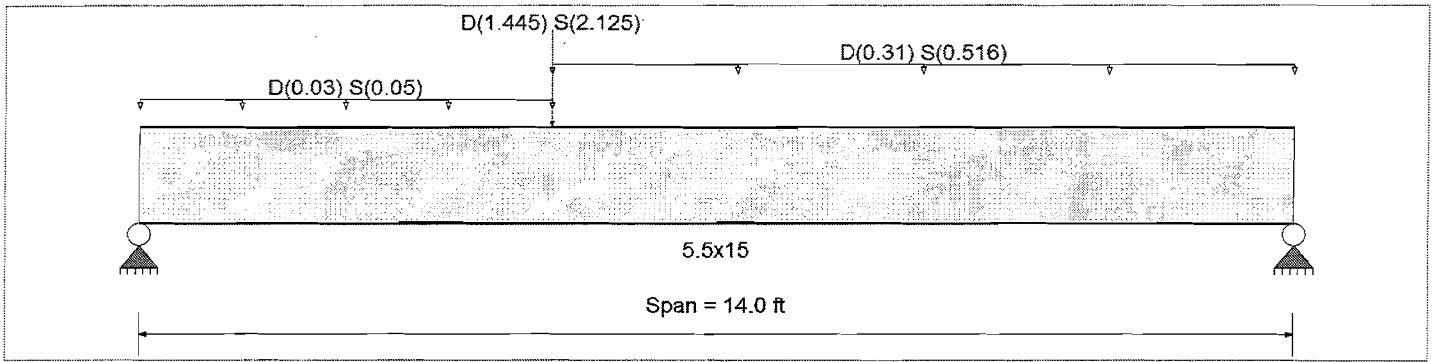
CODE REFERENCES

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 = 0.030, S = 0.050 k/ft, Extent = 0.0 -->> 5.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.310, S = 0.5160 k/ft, Extent = 5.0 -->> 14.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.445, S = 2.125 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.521**: 1	Maximum Shear Stress Ratio	=	**0.321** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	1,438.47 psi		=	97.78 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.285ft	Location of maximum on span	=	12.774 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.188 in	Ratio =	894 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.306 in	Ratio =	549 >=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	5.013	6.391
Overall MINimum	3.064	3.955
D Only	1.949	2.436
+D+L	1.949	2.436
+D+S	5.013	6.391
+D+0.750L	1.949	2.436
+D+0.750L+0.750S	4.247	5.402
+0.60D	1.169	1.462
S Only	3.064	3.955

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 3. Patio Roof Beam

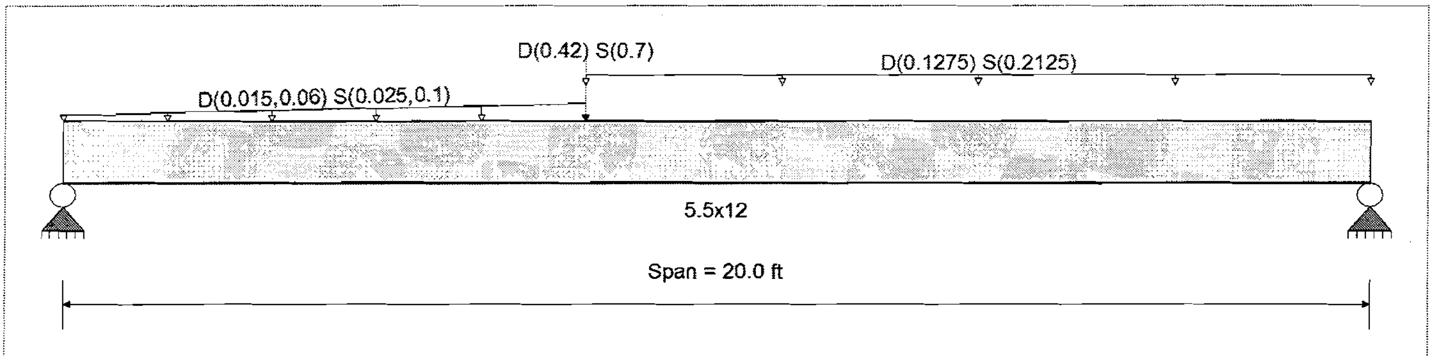
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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.

Load for Span Number 1
Varying Uniform Load : D= 0.0150->0.060, S= 0.0250->0.10 k/ft, Extent = 0.0 -->> 8.0 ft, Trib Width = 1.0 ft

Point Load : D = 0.420, S = 0.70 k @ 8.0 ft
Uniform Load : D = 0.1275, S = 0.2125 k/ft, Extent = 8.0 -->> 20.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.593** : 1	Maximum Shear Stress Ratio	=	**0.237** : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,633.96 psi		=	72.12 psi
	=	2,753.98 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.708 ft	Location of maximum on span	=	19.051 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.553 in	Ratio =	433 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.885 in	Ratio =	271 >=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	2.504	3.496
Overall MINimum	1.565	2.185
D Only	0.939	1.311
+D+L	0.939	1.311
+D+S	2.504	3.496
+D+0.750L	0.939	1.311
+D+0.750L+0.750S	2.113	2.950
+0.60D	0.563	0.787
S Only	1.565	2.185

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 4. Rim Beam at Stair/Great Rm

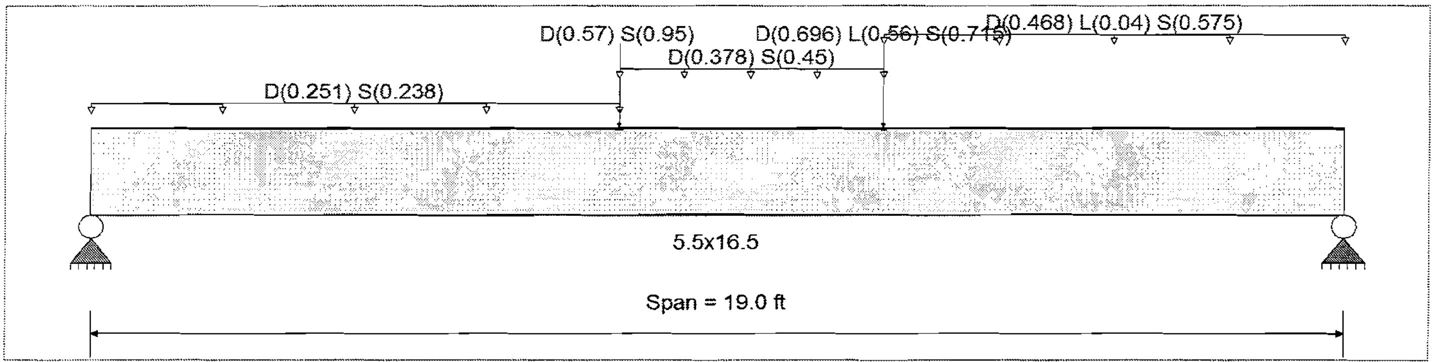
CODE REFERENCES

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.

Load for Span Number 1
- Uniform Load : D = 0.2510, S = 0.2380 k/ft, Extent = 0.0 -->> 8.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.570, S = 0.950 k @ 8.0 ft
- Uniform Load : D = 0.3780, S = 0.450 k/ft, Extent = 8.0 -->> 12.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.6960, L = 0.560, S = 0.7150 k @ 12.0 ft
- Uniform Load : D = 0.4680, L = 0.040, S = 0.5750 k/ft, Extent = 12.0 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.823 : 1**	Maximum Shear Stress Ratio	=	**0.471 : 1**
Section used for this span		**5.5x16.5**	Section used for this span		**5.5x16.5**
	=	2,207.62 psi		=	143.47 psi
	=	2,681.38 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.401 ft	Location of maximum on span	=	17.682 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.429 in	Ratio =	531 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.793 in	Ratio =	287 >=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	7.402	10.054
Overall MINimum	3.911	5.483
D Only	3.491	4.571
+D+L	3.749	5.153
+D+S	7.402	10.054
+D+0.750L	3.685	5.007
+D+0.750L+0.750S	6.618	9.120
+0.60D	2.095	2.742
L Only	0.258	0.582
S Only	3.911	5.483

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Gig Harbor, WA 98335

Wood Beam

Lic. # : KW-06008232

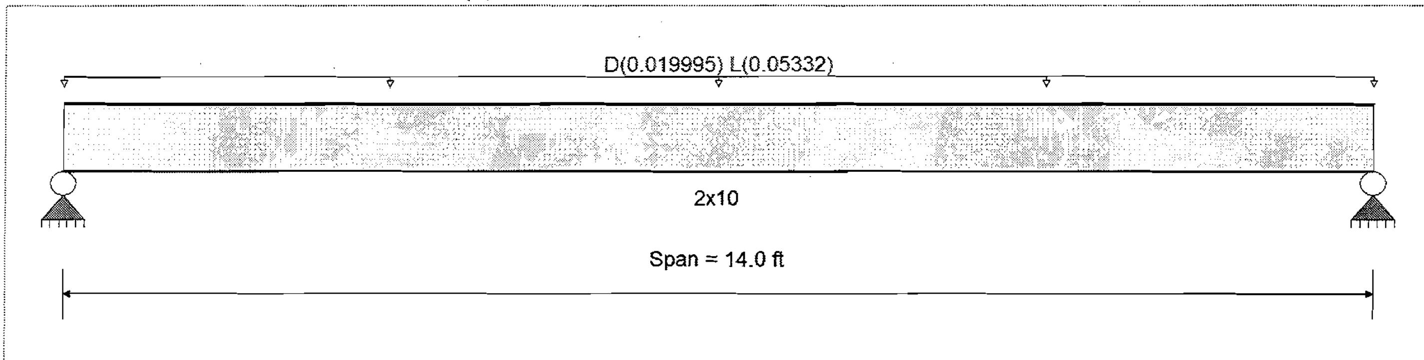
DESCRIPTION: 5. Floor Joist

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 = 0.040 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.937: 1**	Maximum Shear Stress Ratio	=	**0.329 : 1**
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	1,007.67 psi		=	49.41 psi
	=	1,075.25 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.000 ft	Location of maximum on span	=	13.234 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.360 in	Ratio =	466 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.496 in	Ratio =	338 >=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	0.513	0.513
Overall MINimum	0.373	0.373
D Only	0.140	0.140
+D+L	0.513	0.513
+D+S	0.140	0.140
+D+0.750L	0.420	0.420
+D+0.750L+0.750S	0.420	0.420
+0.60D	0.084	0.084
L Only	0.373	0.373
S Only		

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Wood Beam

File: 42xx 89th Ave SE.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

DESCRIPTION: 5a. Floor Joist at Master Bath

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 : Hem-Fir
Wood Grade : No.2

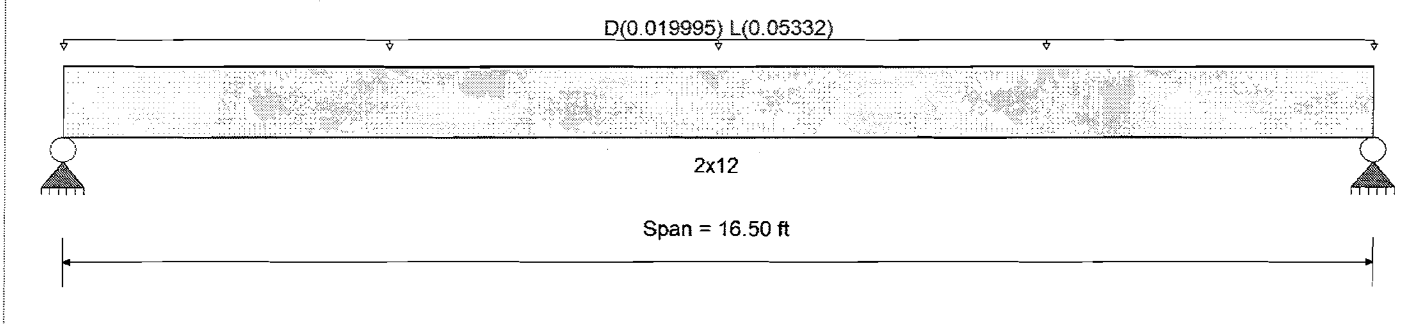
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0 psi
Fb -	850.0 psi
Fc - Prll	1,300.0 psi
Fc - Perp	405.0 psi
Fv	150.0 psi
Ft	525.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	470.0 ksi
Density	26.840 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.968** : 1	Maximum Shear Stress Ratio	=	**0.319** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
	=	946.25 psi		=	47.88 psi
	=	977.50 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.250 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	0.387 in	Ratio =	512 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.532 in	Ratio =	372 >=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	0.605	0.605
Overall MINimum	0.440	0.440
D Only	0.165	0.165
+D+L	0.605	0.605
+D+S	0.165	0.165
+D+0.750L	0.495	0.495
+D+0.750L+0.750S	0.495	0.495
+0.60D	0.099	0.099
L Only	0.440	0.440
S Only		

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Wood Beam

DESCRIPTION: 6. Beam over Great Rm/Kitchen

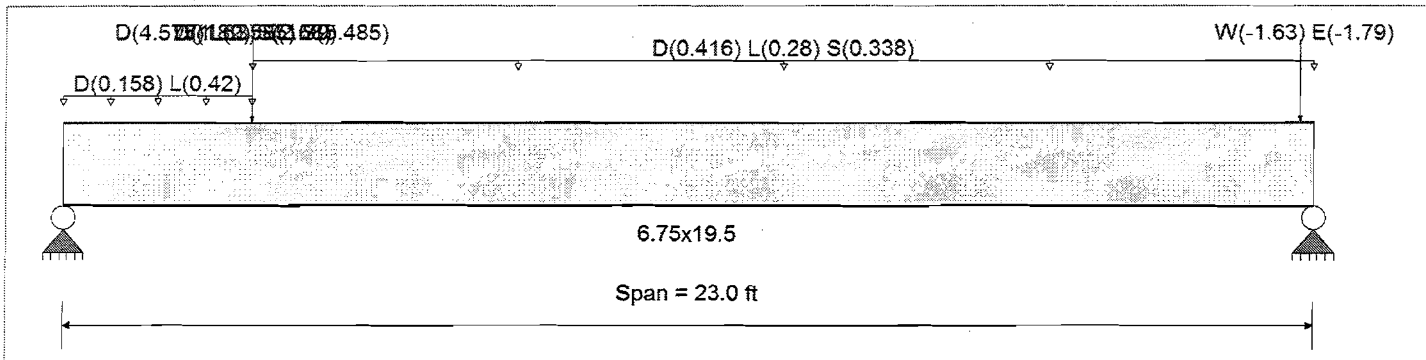
CODE REFERENCES

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 = 0.1580, L = 0.420 k/ft, Extent = 0.0 -->> 3.50 ft, Tributary Width = 1.0 ft
- Point Load : D = 1.820, S = 2.680 k @ 3.50 ft
- Uniform Load : D = 0.4160, L = 0.280, S = 0.3380 k/ft, Extent = 3.50 -->> 23.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 4.575, L = 0.5850, S = 5.485 k @ 3.50 ft
- Point Load : W = 1.630, E = 1.790 k @ 3.50 ft
- Point Load : W = -1.630, E = -1.790 k @ 22.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.903** 1	Maximum Shear Stress Ratio	=	**0.712 : 1**
Section used for this span		**6.75x19.5**	Section used for this span		**6.75x19.5**
	=	2,287.95 psi		=	216.88 psi
	=	2,534.61 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	9.401 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	0.484 in	Ratio =	570 >=360
Max Upward Transient Deflection	-0.044 in	Ratio =	6211 >=360
Max Downward Total Deflection	1.085 in	Ratio =	254 >=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	20.572	12.764
Overall MINimum	-1.498	1.498
D Only	9.372	5.688
+D+L	13.540	9.035
+D+S	19.088	10.728
+D+0.750L	12.498	8.198
+D+0.750L+0.750S	19.785	11.978
+D+0.60W	10.190	4.870
+D-0.60W	8.553	6.507

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DESCRIPTION: 6. Beam over Great Rm/Kitchen

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.70E	10.420	4.640
+D-0.70E	8.323	6.737
+D+0.750L+0.450W	13.112	7.584
+D+0.750L-0.450W	11.884	8.812
+D+0.750L+0.750S+0.450W	20.399	11.364
+D+0.750L+0.750S-0.450W	19.172	12.592
+D+0.750L+0.750S+0.5250E	20.572	11.191
+D+0.750L+0.750S-0.5250E	18.999	12.764
+0.60D+0.60W	6.441	2.595
+0.60D-0.60W	4.804	4.232
+0.60D+0.70E	6.672	2.364
+0.60D-0.70E	4.574	4.462
L Only	4.169	3.346
S Only	9.717	5.039
W Only	1.364	-1.364
-W	-1.364	1.364
E Only	1.498	-1.498
E Only * -1.0	-1.498	1.498

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 6a. Header at Nook

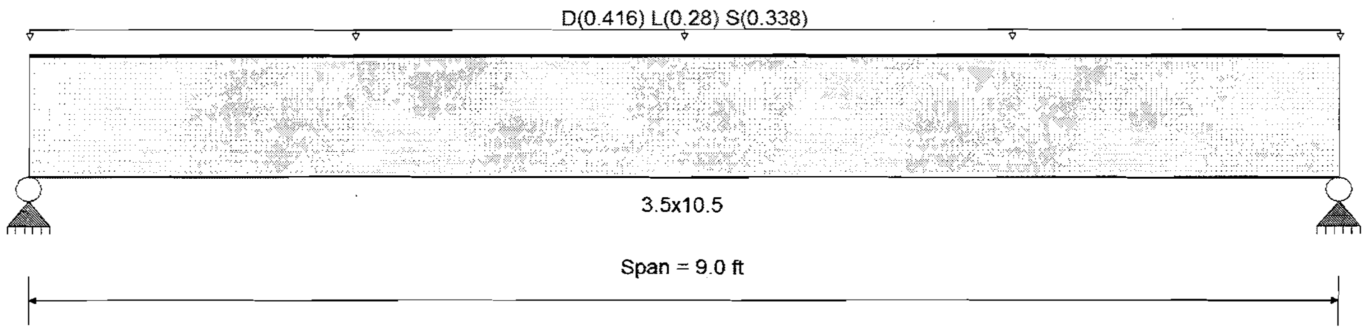
CODE REFERENCES

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 = 0.4160, L = 0.280, S = 0.3380 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.602** 1	Maximum Shear Stress Ratio	=	**0.429** : 1
Section used for this span		**3.5x10.5**	Section used for this span		**3.5x10.5**
	=	1,661.56 psi		=	130.88 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.146 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.083 in	Ratio =	1307 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.215 in	Ratio =	502 >=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	3.958	3.958
Overall MINimum	1.521	1.521
D Only	1.872	1.872
+D+L	3.132	3.132
+D+S	3.393	3.393
+D+0.750L	2.817	2.817
+D+0.750L+0.750S	3.958	3.958
+0.60D	1.123	1.123
L Only	1.260	1.260
S Only	1.521	1.521

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Wood Beam

DESCRIPTION: 7. Rim Beam at Master Shower

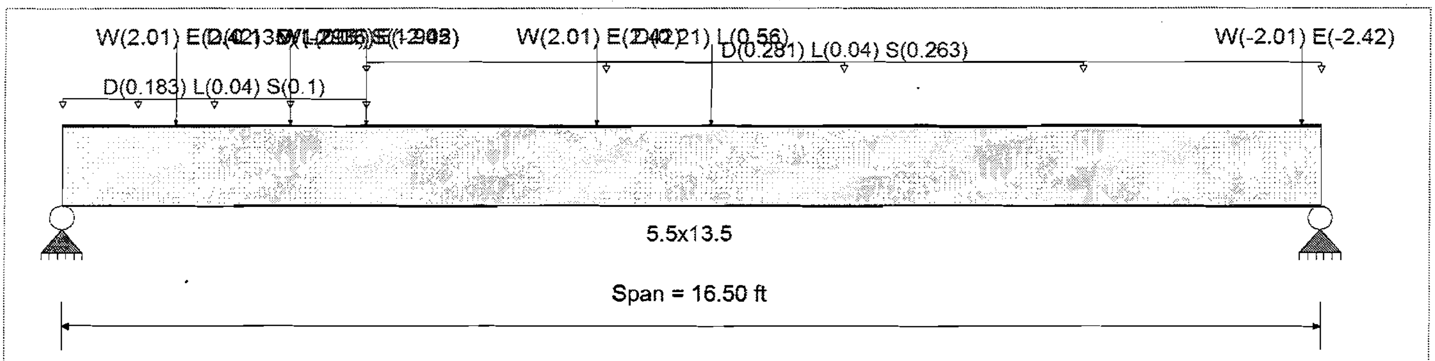
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.1830, L = 0.040, S = 0.10 k/ft, Extent = 0.0 -->> 4.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.2810, L = 0.040, S = 0.2630 k/ft, Extent = 4.0 -->> 16.50 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.1350, L = 0.360 k @ 3.0 ft
- Point Load : D = 1.295, S = 1.905 k @ 4.0 ft
- Point Load : D = 0.210, L = 0.560 k @ 8.50 ft
- Point Load : W = 2.010, E = 2.420 k @ 1.50 ft, (Shear Wall Overturning)
- Point Load : W = -2.010, E = -2.420 k @ 4.0 ft, (Shear Wall Overturning)
- Point Load : W = 2.010, E = 2.420 k @ 7.0 ft, (Shear Wall Overturning)
- Point Load : W = -2.010, E = -2.420 k @ 16.250 ft, (Shear Wall Overturning)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.673** : 1	Maximum Shear Stress Ratio	=	**0.423** : 1
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	2,582.90psi		=	179.29 psi
	=	3,840.00psi		=	424.00 psi
Load Combination		+1.119D+0.750L+0.750S+1.575E	Load Combination		+1.119D+0.750L+0.750S+1.575E
Location of maximum on span	=	7.046ft	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	0.302 in	Ratio =	655 >=360
Max Upward Transient Deflection	-0.101 in	Ratio =	1957 >=360
Max Downward Total Deflection	0.663 in	Ratio =	298 >=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	7.024	6.050
Overall MINimum	-1.723	1.723
D Only	3.167	2.717
+D+L	4.063	3.401
+D+S	6.207	5.270
+D+0.750L	3.839	3.230

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DESCRIPTION: 7. Rim Beam at Master Shower

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S	6.119	5.145
+D+0.60W	4.026	1.859
+D-0.60W	2.308	3.576
+D+0.70E	4.373	1.511
+D-0.70E	1.961	3.924
+D+0.750L+0.450W	4.483	2.586
+D+0.750L-0.450W	3.195	3.874
+D+0.750L+0.750S+0.450W	6.763	4.501
+D+0.750L+0.750S-0.450W	5.475	5.789
+D+0.750L+0.750S+0.5250E	7.024	4.240
+D+0.750L+0.750S-0.5250E	5.214	6.050
+0.60D+0.60W	2.759	0.772
+0.60D-0.60W	1.041	2.489
+0.60D+0.70E	3.107	0.424
+0.60D-0.70E	0.694	2.837
L Only	0.896	0.684
S Only	3.040	2.553
W Only	1.431	-1.431
-W	-1.431	1.431
E Only	1.723	-1.723
E Only * -1.0	-1.723	1.723

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Wood Beam

DESCRIPTION: 8. Rim beam over Garage

CODE REFERENCES

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

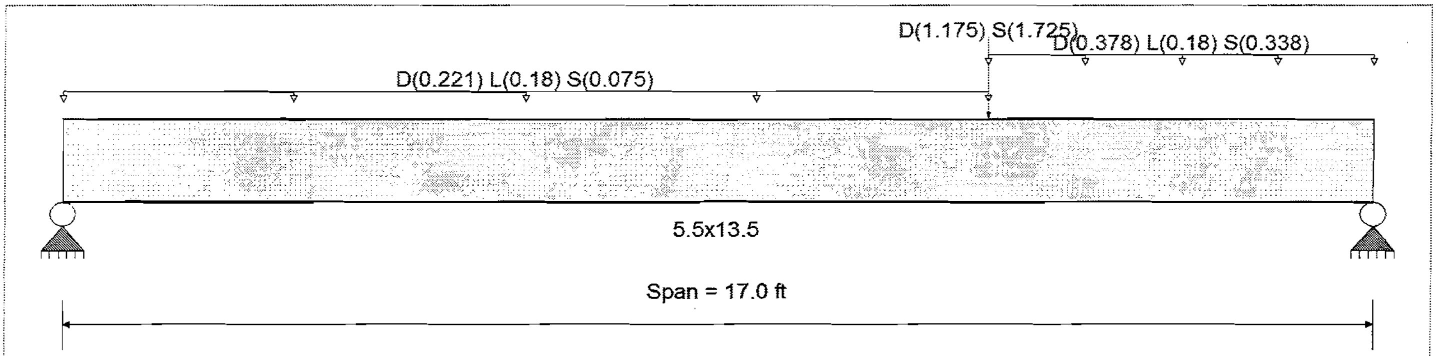
Material Properties

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 = 0.2210, L = 0.180, S = 0.0750 k/ft, Extent = 0.0 -->> 12.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.175, S = 1.725 k @ 12.0 ft
Uniform Load : D = 0.3780, L = 0.180, S = 0.3380 k/ft, Extent = 12.0 -->> 17.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.637**: 1	Maximum Shear Stress Ratio	=	**0.391** : 1
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	1,756.86 psi		=	119.22 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	10.920 ft	Location of maximum on span	=	15.883 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.237 in	Ratio =	859 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.618 in	Ratio =	329 >=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	4.491	6.758
Overall MINimum	1.338	2.977
D Only	2.340	3.377
+D+L	3.870	4.907
+D+S	3.678	6.354
+D+0.750L	3.487	4.525
+D+0.750L+0.750S	4.491	6.758
+0.60D	1.404	2.026
L Only	1.530	1.530
S Only	1.338	2.977

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Wood Beam

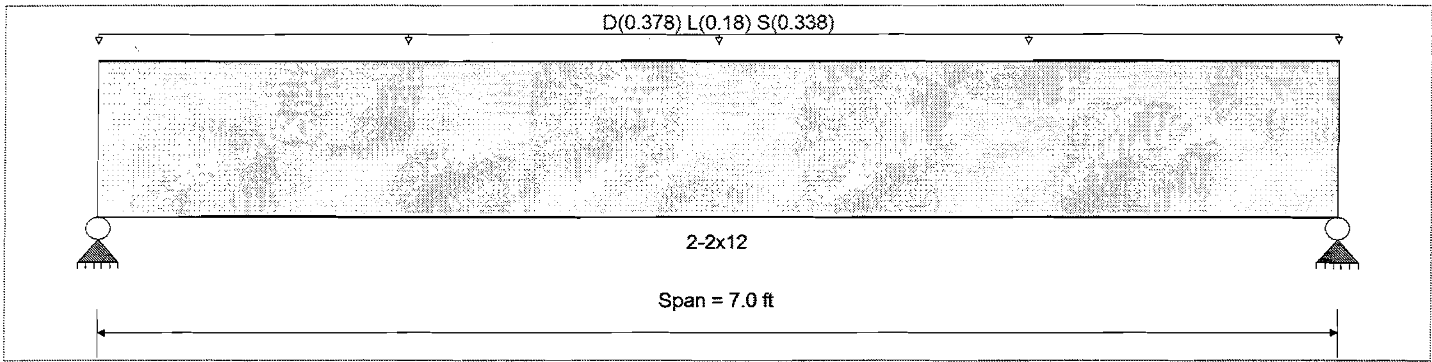
DESCRIPTION: 8a. Rim beam over ADU porch

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				



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.3780, L = 0.180, S = 0.3380 k/ft, Extent = 0.0 -->> 7.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.911**: 1	Maximum Shear Stress Ratio	=	**0.510** : 1
Section used for this span		**2-2x12**	Section used for this span		**2-2x12**
	=	890.28 psi		=	87.90 psi
	=	977.50 psi		=	172.50 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.040 in	Ratio =	2116 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.090 in	Ratio =	933 >=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	2.683	2.683
Overall MINimum	1.183	1.183
D Only	1.323	1.323
+D+L	1.953	1.953
+D+S	2.506	2.506
+D+0.750L	1.796	1.796
+D+0.750L+0.750S	2.683	2.683
+0.60D	0.794	0.794
L Only	0.630	0.630
S Only	1.183	1.183

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 9. Rim Beam over ADU Kichinette

CODE REFERENCES

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

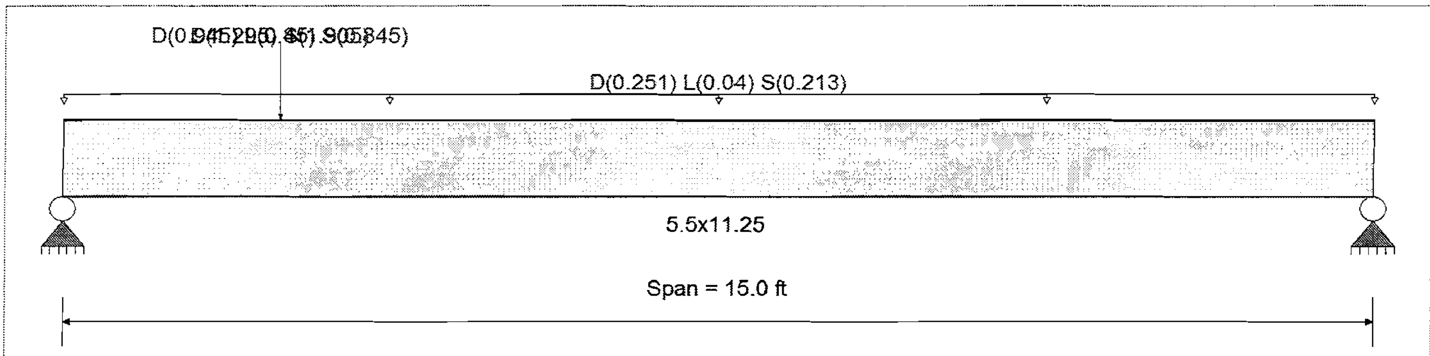
Material Properties

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 = 0.2510, L = 0.040, S = 0.2130 , Tributary Width = 1.0 ft
Point Load : D = 1.295, S = 1.905 k @ 2.50 ft
Point Load : D = 0.9450, L = 0.450, S = 0.8450 k @ 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.751: 1**	Maximum Shear Stress Ratio	=	**0.573 : 1**
Section used for this span		**5.5x11.25**	Section used for this span		**5.5x11.25**
	=	2,072.08psi		=	174.70 psi
	=	2,760.00psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.693ft	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	0.347 in	Ratio =	518 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.705 in	Ratio =	255 >=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	7.638	4.312
Overall MINimum	3.889	2.056
D Only	3.749	2.256
+D+L	4.424	2.631
+D+S	7.638	4.312
+D+0.750L	4.255	2.537
+D+0.750L+0.750S	7.172	4.079
+0.60D	2.250	1.354
L Only	0.675	0.375
S Only	3.889	2.056

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Wood Beam

Lic. # : KW-06008232

File: 42xx 89th Ave SE.ec6

MYERS ENGINEERING

DESCRIPTION: 10. Beam over Garage

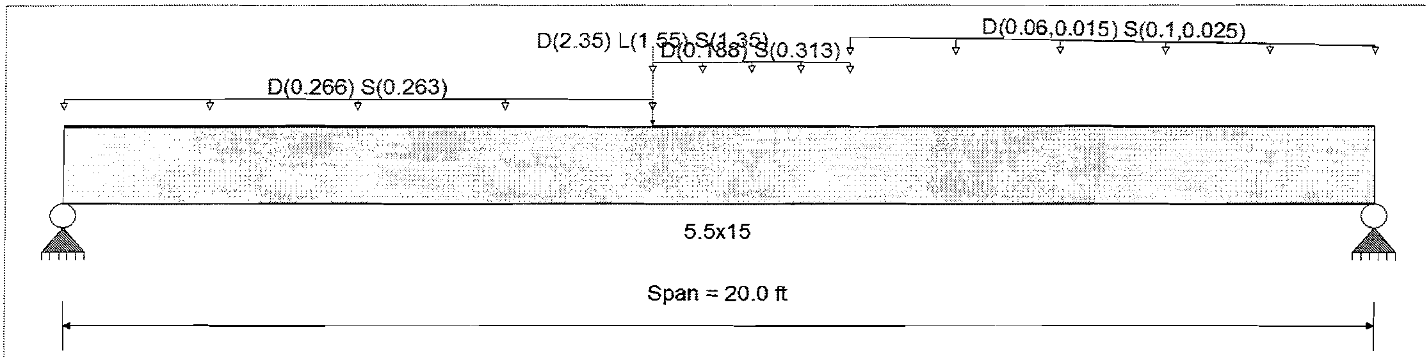
CODE REFERENCES

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.

Load for Span Number 1
- Uniform Load : D = 0.2660, S = 0.2630 k/ft, Extent = 0.0 -->> 9.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 2.350, L = 1.550, S = 1.350 k @ 9.0 ft
- Uniform Load : D = 0.1880, S = 0.3130 k/ft, Extent = 9.0 -->> 12.0 ft, Tributary Width = 1.0 ft
- Varying Uniform Load : D= 0.060->0.0150, S= 0.10->0.0250 k/ft, Extent = 12.0 -->> 20.0 ft, Trib Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.854** 1	Maximum Shear Stress Ratio	=	**0.356** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	2,300.89 psi		=	108.62 psi
	=	2,693.21 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+S
Location of maximum on span	=	8.978 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	0.412 in	Ratio =	582 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.893 in	Ratio =	268 >=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	6.631	4.153
Overall MINimum	3.143	2.013
D Only	3.488	2.120
+D+L	4.340	2.818
+D+S	6.631	4.133
+D+0.750L	4.127	2.643
+D+0.750L+0.750S	6.484	4.153
+0.60D	2.093	1.272
L Only	0.853	0.698
S Only	3.143	2.013

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Wood Beam

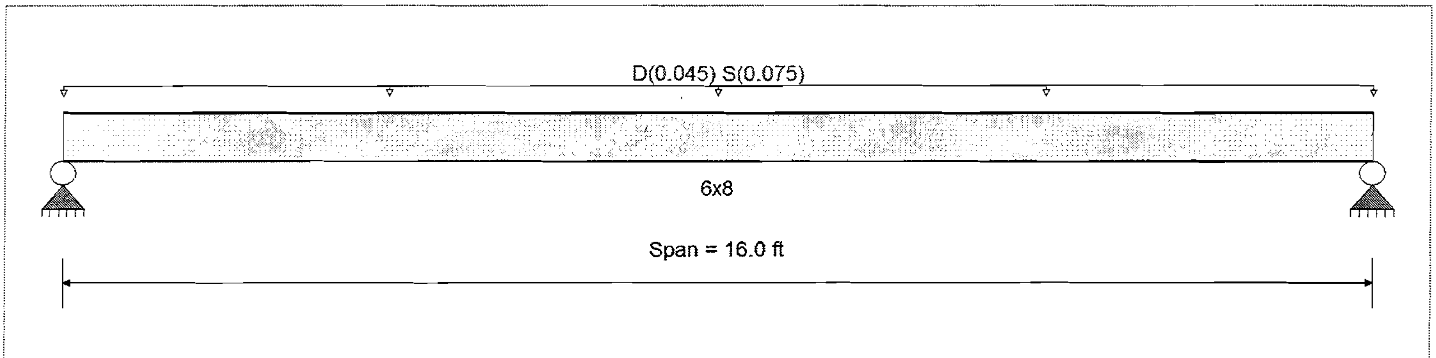
DESCRIPTION: 11. Garage Door Header

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 +	875 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species	: Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	170 psi		
		Ft	425 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 = 0.0450, S = 0.0750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.888: 1	Maximum Shear Stress Ratio	=	0.166 : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
	=	893.67 psi		=	32.36 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.443 in	Ratio =	433 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.708 in	Ratio =	271 >=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	0.960	0.960
Overall MINimum	0.600	0.600
D Only	0.360	0.360
+D+L	0.360	0.360
+D+S	0.960	0.960
+D+0.750L	0.360	0.360
+D+0.750L+0.750S	0.810	0.810
+0.60D	0.216	0.216
S Only	0.600	0.600

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Wood Beam

File: 42xx 89th Ave SE.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

DESCRIPTION: 12. Floor beam over Dining Rm

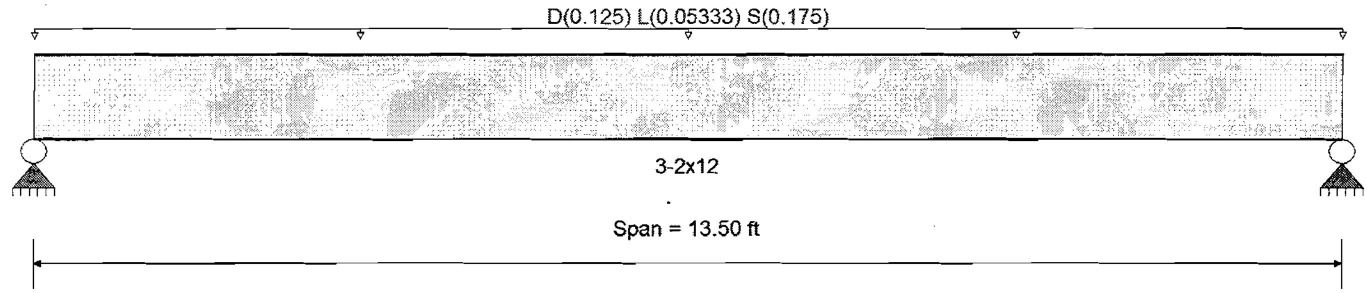
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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



Applied Loads

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

Uniform Load : D = 0.1250, L = 0.05333, S = 0.1750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.884** : 1	Maximum Shear Stress Ratio	=	**0.300** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
	=	864.00 psi		=	51.68 psi
	=	977.50 psi		=	172.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.750 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	0.190 in	Ratio =	854 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.325 in	Ratio =	498 >=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	2.025	2.025
Overall MINimum	1.181	1.181
D Only	0.844	0.844
+D+L	1.204	1.204
+D+S	2.025	2.025
+D+0.750L	1.114	1.114
+D+0.750L+0.750S	2.000	2.000
+0.60D	0.506	0.506
L Only	0.360	0.360
S Only	1.181	1.181

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Wood Beam

File: 42xx 89th Ave SE.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

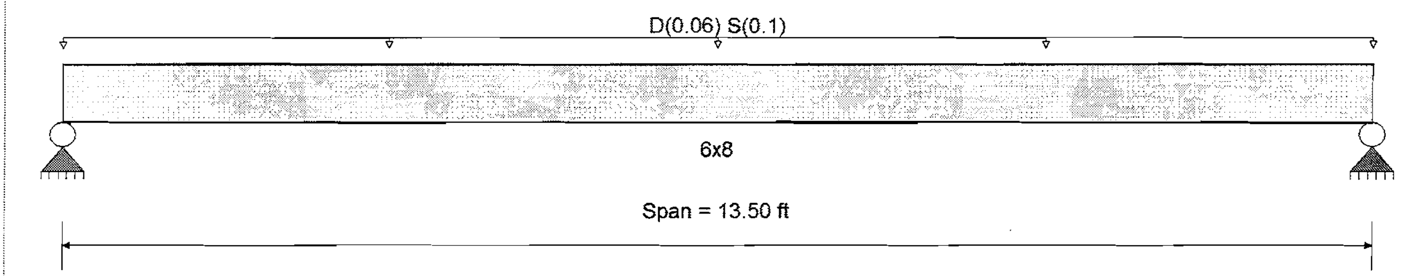
DESCRIPTION: 13. Roof beam at Front Porch

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 +	875 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species	: Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	170 psi		
		Ft	425 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 = 0.060, S = 0.10 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.843: 1**	Maximum Shear Stress Ratio	=	**0.183 : 1**
Section used for this span		**6x8**	Section used for this span		**6x8**
	=	848.29 psi		=	35.83 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.750 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	0.299 in	Ratio =	541 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.478 in	Ratio =	338 >=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	1.080	1.080
Overall MINimum	0.675	0.675
D Only	0.405	0.405
+D+L	0.405	0.405
+D+S	1.080	1.080
+D+0.750L	0.405	0.405
+D+0.750L+0.750S	0.911	0.911
+0.60D	0.243	0.243
S Only	0.675	0.675

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Wood Beam

Lic. # : KW-06008232

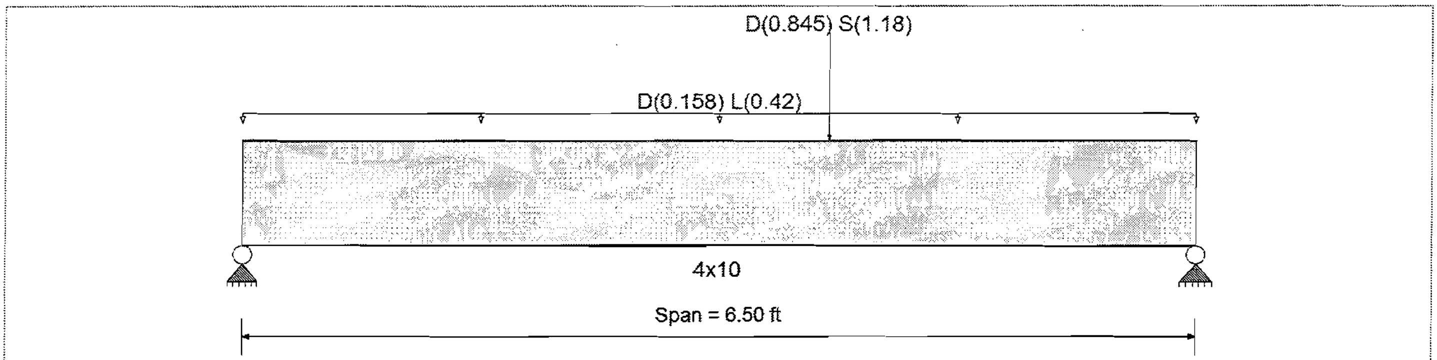
DESCRIPTION: 14. Header at Dining

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 = 0.1580, L = 0.420 , Tributary Width = 1.0 ft
Point Load : D = 0.8450, S = 1.180 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.972** 1	Maximum Shear Stress Ratio	=	**0.504** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,207.41 psi		=	90.80 psi
	=	1,242.00 psi		=	180.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+L
Location of maximum on span	=	3.985 ft	Location of maximum on span	=	5.741 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.046 in	Ratio =	1697 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.095 in	Ratio =	821 >=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	2.204	2.602
Overall MINimum	0.454	0.726
D Only	0.839	1.034
+D+L	2.204	2.399
+D+S	1.292	1.760
+D+0.750L	1.862	2.057
+D+0.750L+0.750S	2.203	2.602
+0.60D	0.503	0.620
L Only	1.365	1.365
S Only	0.454	0.726

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Wood Beam

File: 42xx 89th Ave SE.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

DESCRIPTION: 15. Rim Joist at top of Stair

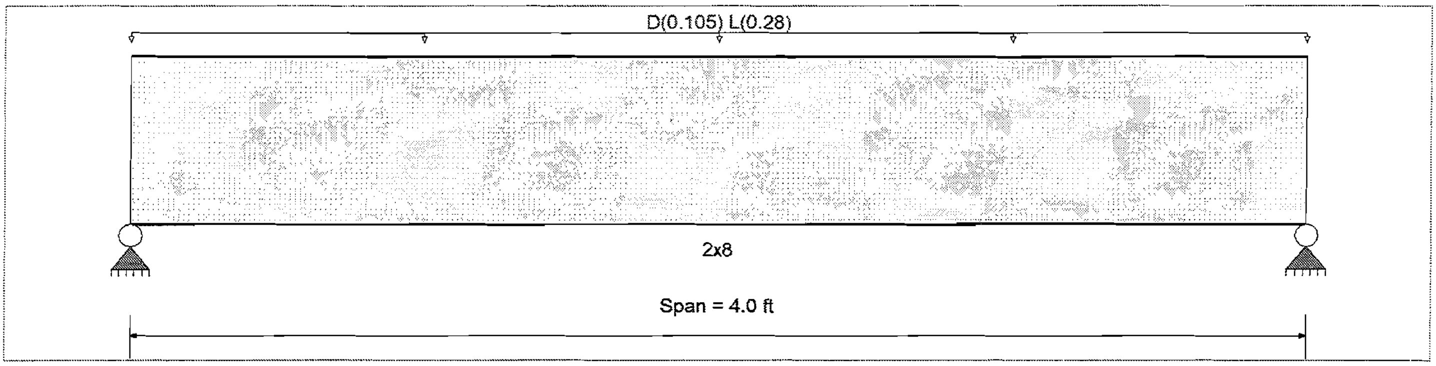
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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



Applied Loads

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

Uniform Load : D = 0.1050, L = 0.280 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.689** 1	Maximum Shear Stress Ratio	=	**0.496** : 1
Section used for this span		**2x8**	Section used for this span		**2x8**
	=	703.16 psi		=	74.42 psi
	=	1,020.00 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.000 ft	Location of maximum on span	=	3.401 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.026 in	Ratio =	1832 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.036 in	Ratio =	1332 >=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	0.770	0.770
Overall MINimum	0.560	0.560
D Only	0.210	0.210
+D+L	0.770	0.770
+D+S	0.210	0.210
+D+0.750L	0.630	0.630
+D+0.750L+0.750S	0.630	0.630
+0.60D	0.126	0.126
L Only	0.560	0.560
S Only		

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Wood Beam

Lic. # : KW-06008232
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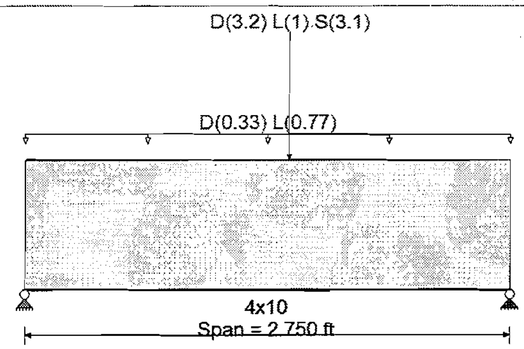
DESCRIPTION: 16. Crawl Beam

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 +	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 = 0.330, L = 0.770 , Tributary Width = 1.0 ft
Point Load : D = 3.20, L = 1.0, S = 3.10 k @ 1.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.990 : 1**	Maximum Shear Stress Ratio	=	**0.890 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,230.18 psi		=	184.32 psi
	=	1,242.00 psi		=	207.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	1.495 ft	Location of maximum on span	=	1.987 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.006 in	Ratio =	5281 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.016 in	Ratio =	2085 >=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	4.100	4.671
Overall MINimum	1.409	1.691
D Only	1.908	2.199
+D+L	3.422	3.803
+D+S	3.317	3.890
+D+0.750L	3.043	3.402
+D+0.750L+0.750S	4.100	4.671
+0.60D	1.145	1.320
L Only	1.513	1.604
S Only	1.409	1.691

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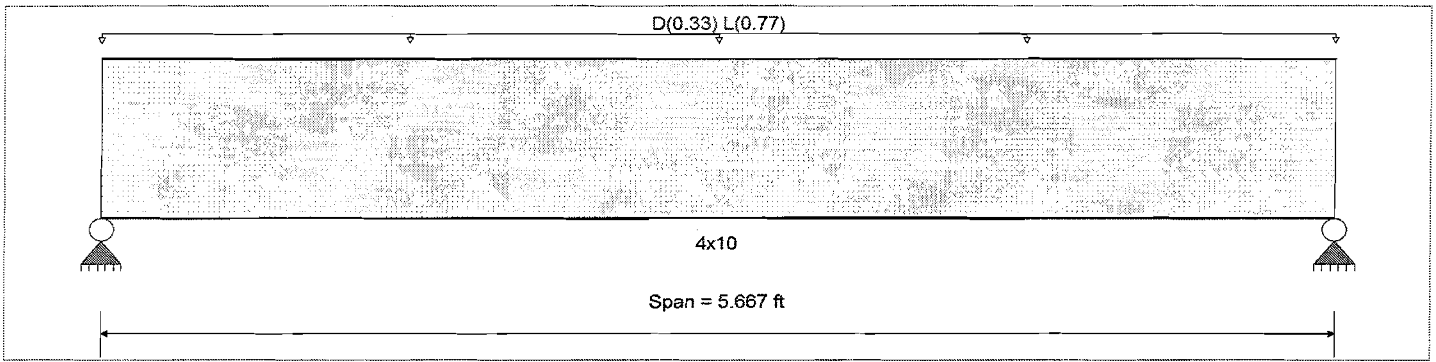
DESCRIPTION: 16a. Crawl Beam w/o Pt. Load

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 = 0.330, L = 0.770 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.983** 1	Maximum Shear Stress Ratio	=	**0.586** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,061.67 psi		=	105.41 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.834 ft	Location of maximum on span	=	4.902 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.049 in	Ratio =	1397 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.070 in	Ratio =	978 >=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	3.117	3.117
Overall MINimum	2.182	2.182
D Only	0.935	0.935
+D+L	3.117	3.117
+D+S	0.935	0.935
+D+0.750L	2.571	2.571
+D+0.750L+0.750S	2.571	2.571
+0.60D	0.561	0.561
L Only	2.182	2.182
S Only		

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PROJECT : **42xx 89th AVE SE**

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)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

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)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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PROJECT : **42xx 89th AVE SE**

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)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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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)

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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)