MYERS ENGINEERING

LATERAL ANALYSIS &
GRAVITY CALCULATIONS

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Mark Myers, PE
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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: RKK Lot 3
3404 72nd Place Southeast
Mercer Island, WA

July 8, 2020

2015 INTERNATIONAL BUILDING CODE
110 MPH WIND, EXPOSURE C, K_{zt} = 1.65
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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DESIGN LOADS:

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

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

WOODS :

WOOD TYPE:

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

GLUED-LAMINATED (GLB) BEAM & HEADER.
Fb=2,400 PSI, Fv=165 PSI, Fc (Perp) =650 PSI, E=1,800,000 PSI.

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

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

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

TRUSSES:

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

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

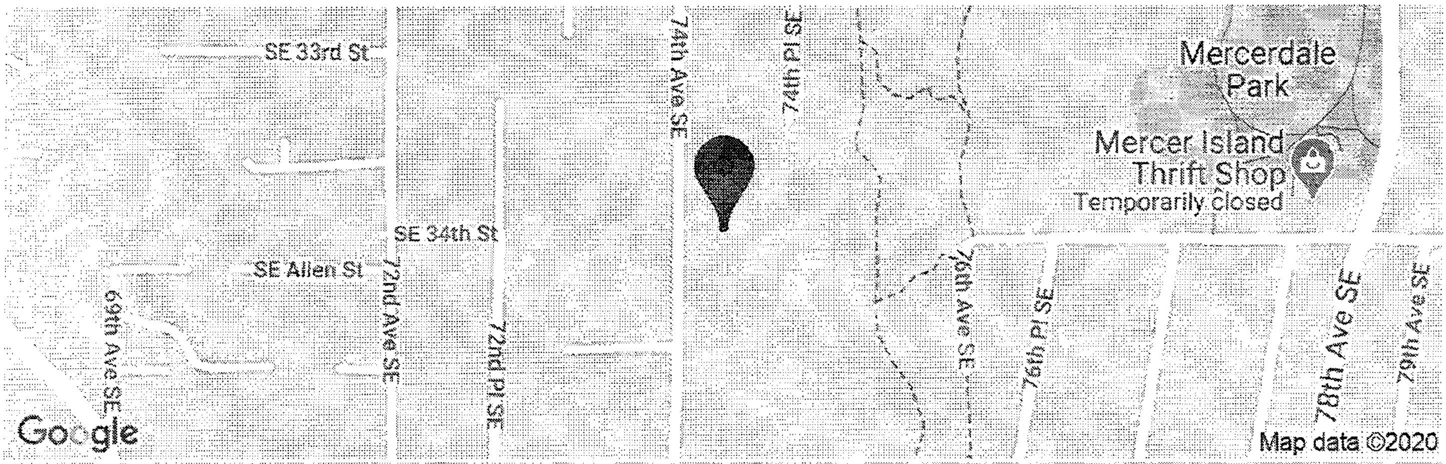
ENGINEERED I-JOISTS

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

OSHPD

3404 72nd Place SE

Latitude, Longitude: 47.58, -122.24



Date	7/2/2020, 5:08:52 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.394	MCE_R ground motion. (for 0.2 second period)
S_1	0.536	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.394	Site-modified spectral acceleration value
S_{M1}	0.804	Site-modified spectral acceleration value
S_{DS}	0.929	Numeric seismic design value at 0.2 second SA
S_{D1}	0.536	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.574	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.574	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.394	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.453	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.8	Factored deterministic acceleration value. (0.2 second)
S1RT	0.536	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.574	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.163	Factored deterministic acceleration value. (1.0 second)
PGAd	1.075	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.959	Mapped value of the risk coefficient at short periods
C_{R1}	0.934	Mapped value of the risk coefficient at a period of 1 s

LATERAL ANALYSIS :
BASED ON 2015 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 2015 IBC Section 1613.1
LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT ABOVE GRADE.

Seismic Design Data:

$I_e := 1.0$ (ASCE 7-10 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 7-10 Table 12.2-1)

$S_s := 1.394$ $S_1 := 0.536$ $S_{ms} := 1.394$ $S_{m1} := 0.804$

Equation 16-39 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.93$ Equation 16-40 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.54$

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

Roof Slope Adjustment Factor: $S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{8}{12}\right)\right)} = 1.2$ $S_b := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{4}{12}\right)\right)} = 1.05$

Plan Area for Each Level:

$A_1 := 2040\text{ft}^2 \cdot S_a$ (Upper Roof) $A_{2a} := 1862\text{ft}^2$ (Framed Floor) $A_{2b} := 1070\text{ft}^2 \cdot S_b$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(41\text{ft}) + 2(48\text{ft})$ (Main Floor) $P_2 := 2(44\text{ft}) + 2(62\text{ft})$ (Lower Floor)

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

Weight of Structure at Each Level:

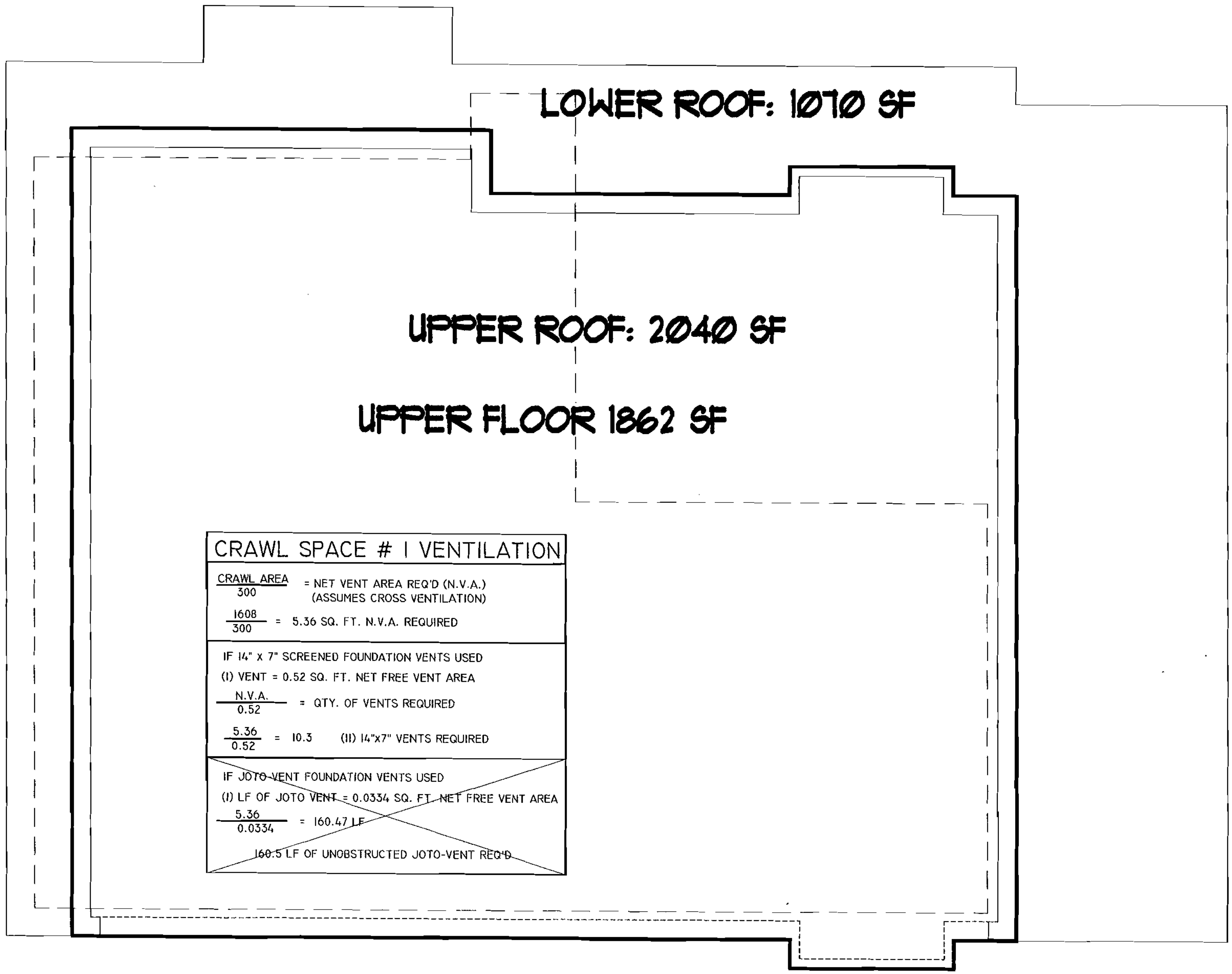
Story Weight at Upper Floor:

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

Story Weight at Main Floor:

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

$$W := w_1 + w_2 = 113568.81\ \text{lb}$$


LOWER ROOF: 1070 SF
UPPER ROOF: 2040 SF
UPPER FLOOR 1862 SF
CRAWL SPACE # 1 VENTILATION
CRAWL AREA / 300 = NET VENT AREA REQ'D (N.V.A.)
(ASSUMES CROSS VENTILATION)
1608 / 300 = 5.36 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
5.36 / 0.52 = 10.3 (11) 14"x7" VENTS REQUIRED
IF JOTO-VENT FOUNDATION VENTS USED
(1) LF OF JOTO VENT = 0.0334 SQ. FT. NET FREE VENT AREA
5.36 / 0.0334 = 160.47 LF
160.5 LF OF UNOBSTRUCTED JOTO-VENT REQ'D

Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE7-10 Table 12.8-2) $\quad h_n := 24$ (Structural Height per ASCE7-10 Sect. 11.2)

$T_a := C_t \cdot h_n^{\chi} = 0.22$ (ASCE7-10 Eq. 12.8-7) $\quad T_L := 6$ (per ASCE7-10 Fig. 22-12)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.38$ (ASCE7-10 Eq. 12.8-3)

C_s shall not be less than: $0.044 S_{DS} \cdot I_e = 0.04$ (ASCE7-10 Eq. 12.8-5)

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.14$$

Total Base Shear: $V_E := C_s \cdot W = 16237.43\,\text{lb}$

Vertical Shear distribution at each level:

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

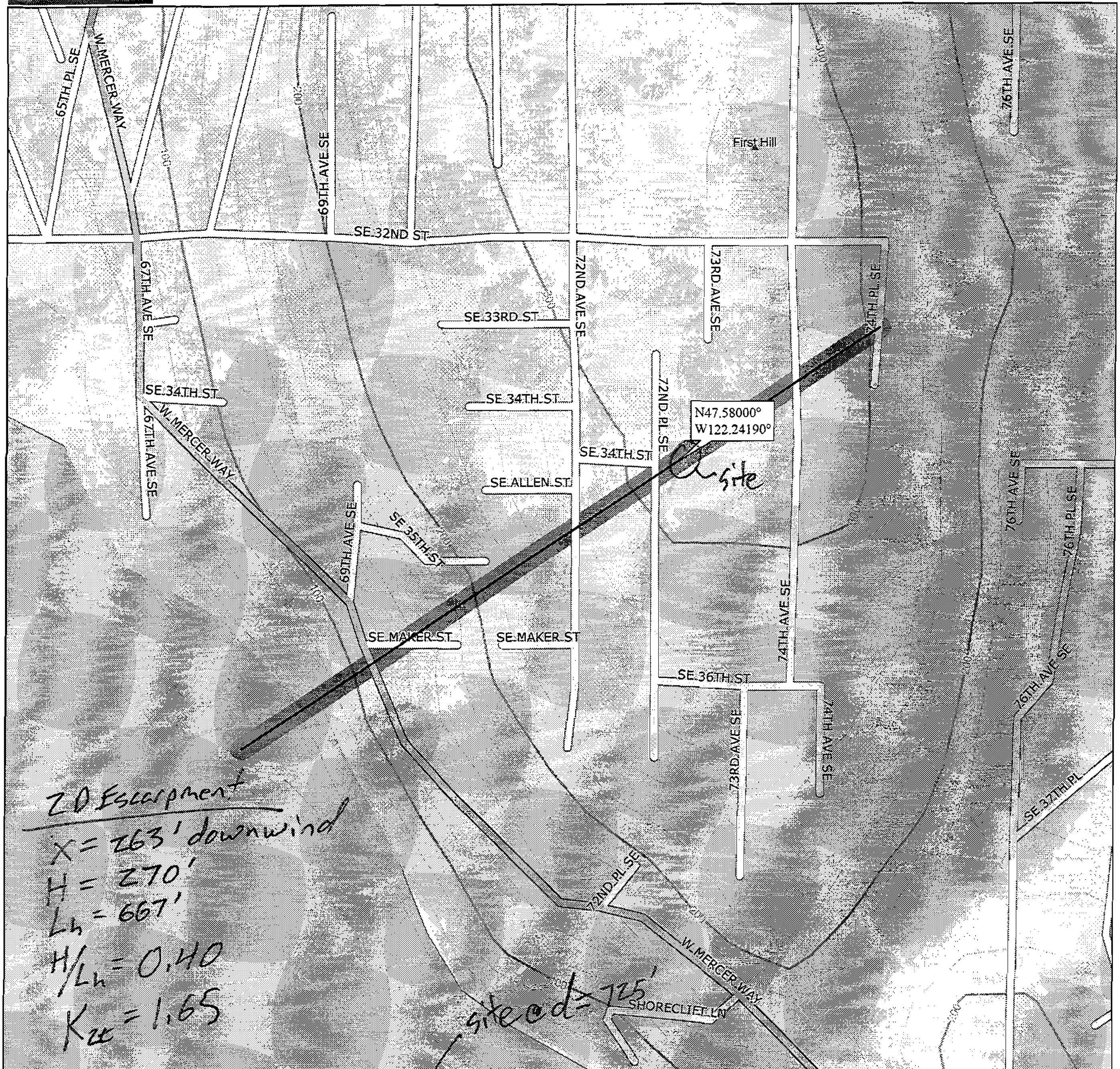
$h_1 := 20\text{ft}$ $\quad h_2 := 10\text{ft}$ (Height from base to level x)

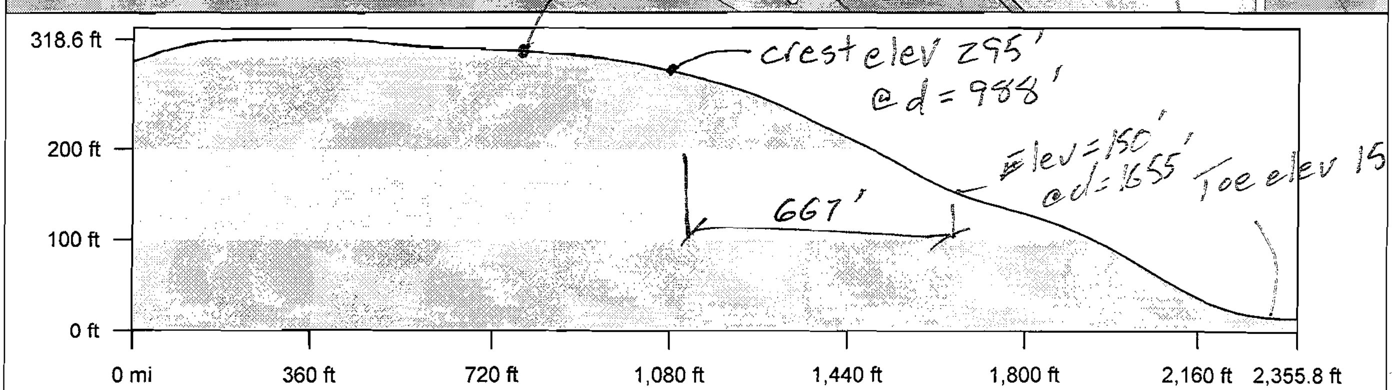
$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.58$$

$F_1 := C_{v1} \cdot V_E = 9417.9\,\text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.42$$

$F_2 := C_{v2} \cdot V_E = 6819.52\,\text{lb}$ Story Shear at Main Floor





Lin Dist: 2,320.5 ft	Terr Dist: 2,355.8 ft	Elev Gain: -280.4 ft	Avg Grade: 14
Climb Elev: 24.0 ft	Desc Elev: 304.4 ft	Max. Elev: 318.6 ft	Min. Elev: 14.4 ft
Climb Dist: 308.1 ft	Desc Dist: 2,047.6 ft		

Scale 1 : 5,600

1" = 466.7 ft

Data Zoom 15-2

TN

MN (15.2°E)

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WIND DESIGN

Use analytical procedure of ASCE 7-10 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 C (ASCE7-10 Sect. 26.7.3)

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

$x := 263ft$ $H := 270\cdot ft$ $L_h := 667ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.85\left(\frac{H}{L_h}\right) = 0.34 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.9 \qquad K_3 := e^{\frac{(-\gamma\cdot z)}{L_h}} = 0.91 \qquad K_{zt} := \left(1 + K_1\cdot K_2\cdot K_3\right)^2 = 1.65$$

$G := 0.85$ Gust Effect Factor (ASCE7-10 Sect. 26.9.1)

Building is an Enclosed Building as per ASCE7-10 Sect. 26.10

$GC_{pi} := .18$ +/- Internal Pressure Coefficients (ASCE7-10 Table 26.11-1)

Velocity Pressure Exposure Coefficient (Table 27.3-1):

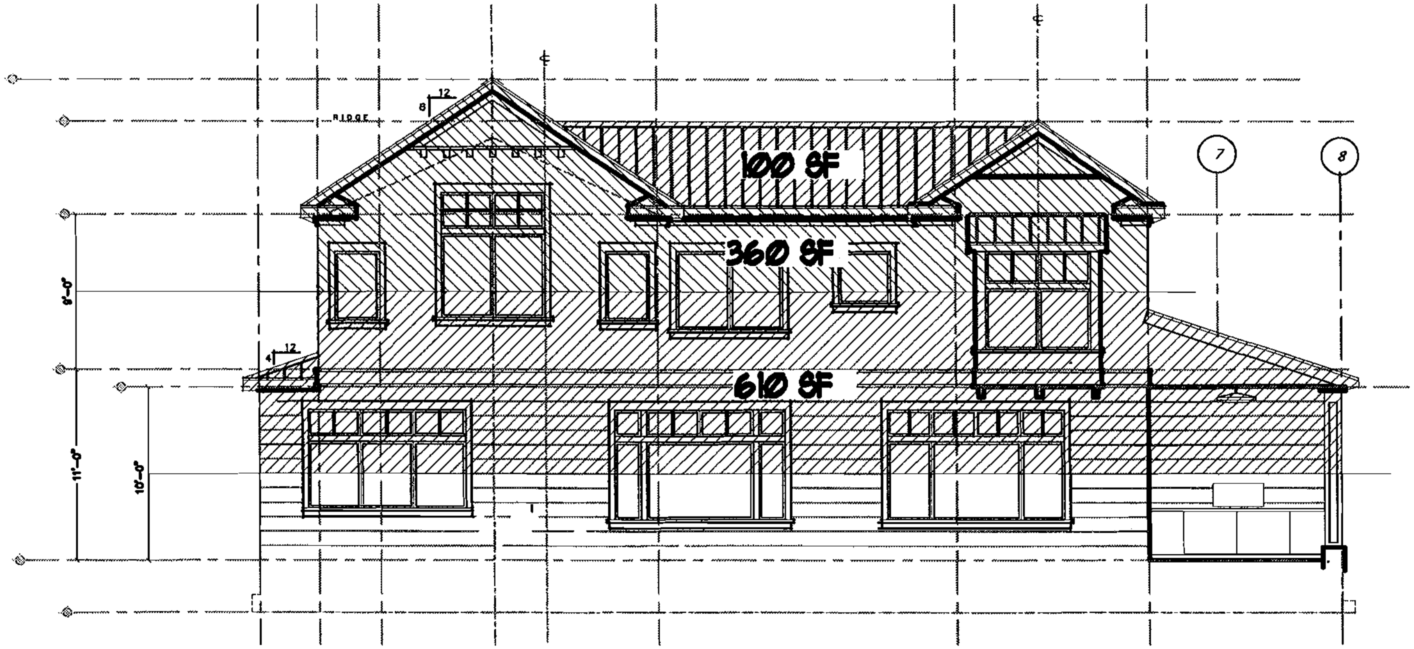
$z_g := 900ft$ $\alpha := 9.5$ (per ASCE7-10 Table 26.9-1 based on Exposure Category)
z_g=1200ft, α=7.0 (Exp B), z_g=900ft, α=9.5 (Exp C), z_g=700ft, α=11.5 (Exp D)

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

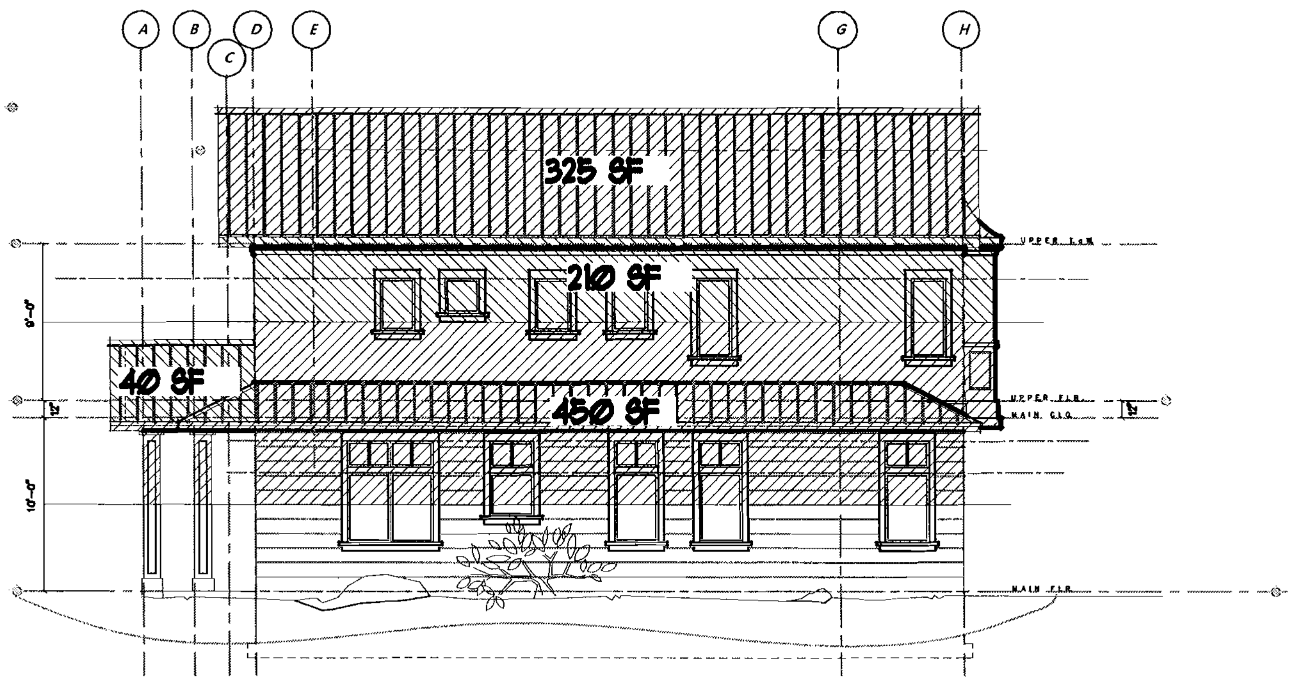
$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.9 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.85 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.94$$

External Pressure Coefficients w/ Roof Pitch = 3/12 (14 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.4-1

Front to Back:		Side to Side:	
$L_{fb} := 41ft$ $B_{fb} := 48ft$ $\frac{L_{fb}}{B_{fb}} = 0.85$ $\frac{h}{L_{fb}} = 0.59$		$L_{ss} := 48ft$ $B_{ss} := 41ft$ $\frac{L_{ss}}{B_{ss}} = 1.17$ $\frac{h}{L_{ss}} = 0.5$	
$C_{pf1} := .8$	Windward Wall	$C_{ps1} := .8$	Windward Wall
$C_{pf2} := -0.18$	Windward Roof	$C_{ps2} := 0.3$	Windward Roof
$C_{pf3} := -.5$	Leeward Roof	$C_{ps3} := -.6$	Leeward Roof
$C_{pf4} := -.5$	Leeward Wall	$C_{ps4} := -.47$	Leeward Wall



SOUTH ELEVATION



WEST ELEVATION

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot V^2 = 39.12$ $\quad q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot V^2 = 36.82$ $\quad q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 = 40.65$

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 Wall Both Directions $\quad p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 26.6\,ft^{-2} \cdot lb$ $\quad p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 25.04\,ft^{-2} \cdot lb$

Windward Roof Front to Back $\quad p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -6.22\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $\quad p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -17.28\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $\quad p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -17.28\,ft^{-2} \cdot lb$

Windward Roof Side to Side $\quad p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 10.37\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $\quad p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -20.73\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $\quad p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf \doteq -16.24\,ft^{-2} \cdot lb$

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:

$p_{wr1} - p_{lr1} = 11.06\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw1} = 43.88\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw1} = 42.31\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 31.1\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw2} = 42.84\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw2} = 41.28\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})100ft^2 + (p_{ww1} - p_{lw1}) \cdot 360 \cdot ft^2 = 16900.81\,lb$$

Wind Pressure at Main Floor (Front to Back):

$$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 0ft^2 + (p_{ww1} - p_{lw1}) \cdot 610ft^2 = 26764.05\,lb$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 325ft^2 + (p_{ww1} - p_{lw2}) \cdot 210ft^2 = 19102.36\,lb$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 40ft^2 + (p_{ww2} - p_{lw2}) \cdot 450ft^2 = 19817.93\,lb$$

Determine Component & Cladding loads:

Design Wind Pressures $p = q_h[(GC_p) - (GC_{pi})]$ (Equation 30.4-1)

(GC_p) is given in Figures 30.4-1 thru 30.4-7

(GC_{pi}) is given in Table 26.11-1 (See above)

$GC_{p1in} := 0.9$ $\quad GC_{p2in} := 0.9$ $\quad GC_{p3in} := 0.9$ $\quad$ Figure 30.4-2C (θ = 34 degrees)

$GC_{p1out} := -1.0$ $\quad GC_{p2out} := -1.2$ $\quad GC_{p3out} := -1.2$ $\quad GC_{p2oh} := -2.0$ $\quad GC_{p3oh} := -2.0$

$GC_{p4in} := 1.0$ $\quad GC_{p5in} := 1.0$ $\quad$ Figure 30.4-1

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

$p_1 := q_h \cdot \left[\left(GC_{p1out} \right) - \left(GC_{pi} \right) \right] psf$ $\quad p_1 = -47.96\, ft^{-2} \cdot lb$ $\quad$ (Zone 1)

$p_2 := q_h \cdot \left[\left(GC_{p2out} \right) - \left(GC_{pi} \right) \right] psf$ $\quad p_2 = -56.09\, ft^{-2} \cdot lb$ $\quad$ (Zone 2)

$p_3 := q_h \cdot \left[\left(GC_{p3out} \right) - \left(GC_{pi} \right) \right] psf$ $\quad p_3 = -56.09\, ft^{-2} \cdot lb$ $\quad$ (Zone 3)

$p_2 := q_h \cdot \left(\left(GC_{p2oh} \right) \right) psf$ $\quad p_2 = -81.3\, ft^{-2} \cdot lb$ $\quad$ (Zone 2 Overhang)

$p_3 := q_h \cdot \left(\left(GC_{p3oh} \right) \right) psf$ $\quad p_3 = -81.3\, ft^{-2} \cdot lb$ $\quad$ (Zone 3 Overhang)

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

$p_4 := q_h \cdot \left[\left(GC_{p4out} \right) - \left(GC_{pi} \right) \right] psf$ $\quad p_4 = -52.03\, ft^{-2} \cdot lb$ $\quad$ (Zone 4)

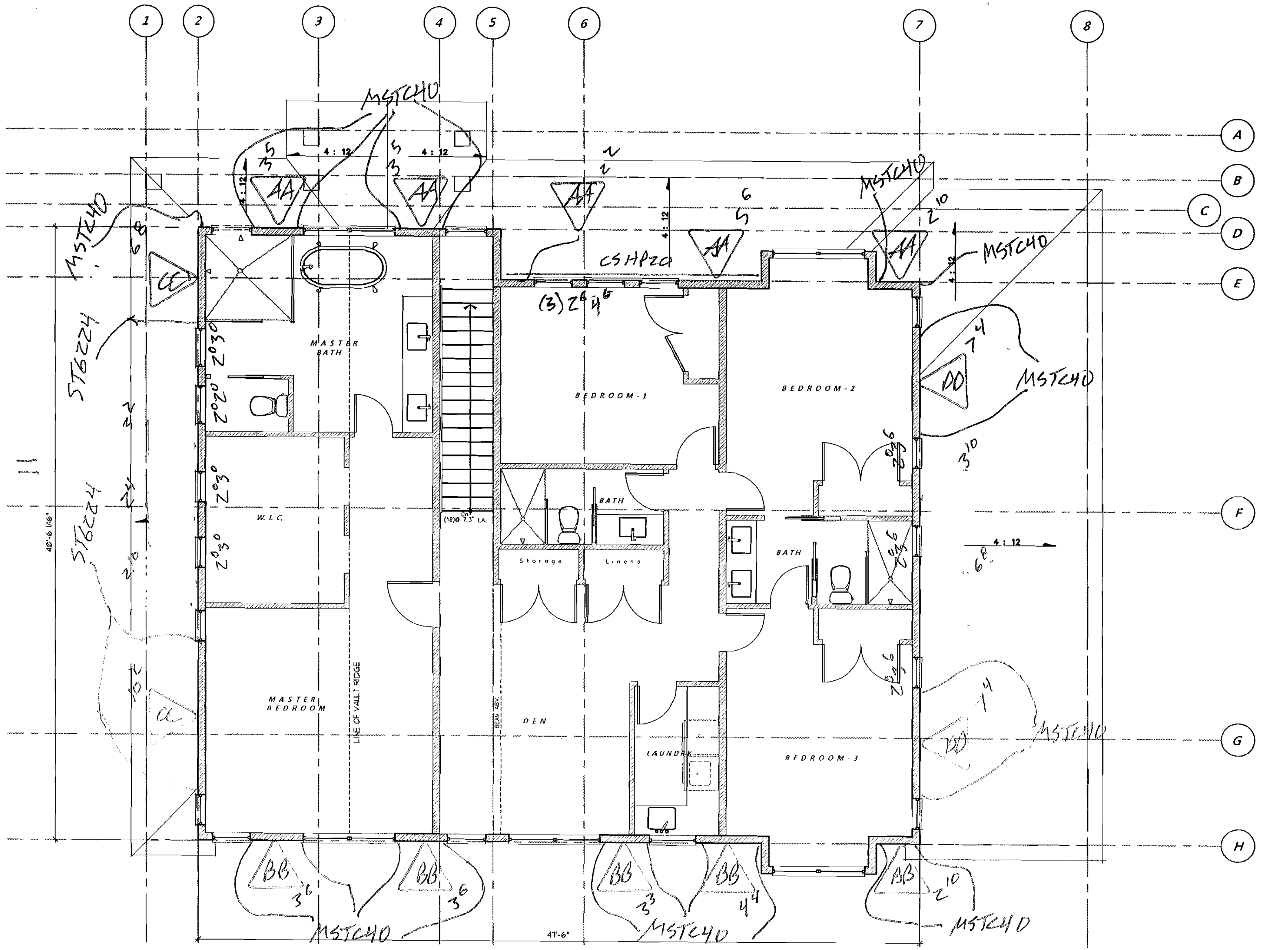
$p_5 := q_h \cdot \left[\left(GC_{p5out} \right) - \left(GC_{pi} \right) \right] psf$ $\quad p_5 = -64.22\, ft^{-2} \cdot lb$ $\quad$ (Zone 5)

Net pressure shall not be less than 16 psf for Components and Cladding (ASCE 7-10 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

$0.1(46ft) = 4.6\, ft$ $\quad 0.4 \cdot h = 9.6\, ft$ $\quad 0.04(46ft) = 1.84\, ft$

Therefore $\quad a := 4.6ft$



1
2
3
4
5
6
7
8
A
B
C
D
E
F
G
H
MSTC40
4 : 12
4 : 12
AA
CSHP20
(3) 2^6 4^6
CC
STG224
MASTER BATH
2^0 3^0
2^0 2^0
W. I. C.
2^0 3^0
2^0 3^0
MASTER BEDROOM
LINE OF VAULT RIDGE
(18) @ 7.3" EA.
BEAM ABV.
DEN
BEDROOM-1
BATH
Storage
Linens
LAUNDRY
BEDROOM-2
DD
BATH
4 : 12
BEDROOM-3
BB
3^6
3^3
4^4
2^10
40'-6 1/16"
47'-6"

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WALL AA:

Story Shear due to Wind: $V_{3W} = 19102.36\,\mathrm{lb}$ Story Shear due to Seismic: $F_1 = 9417.9\,\mathrm{lb}$

Bldg Width in direction of Load: $L_t := 40.5\cdot\mathrm{ft}$ Distance between shear walls: $L_1 := 40.5\cdot\mathrm{ft}$

Shear Wall Length:

$Laa_w := (2\cdot3.42 + 2.17 + 5.5 + 2.83)\mathrm{ft} = 17.34\,\mathrm{ft}$

$$Laa_s := \left[2\cdot3.42\left(\frac{6.83}{9}\right) + 2.17\left(\frac{4.33}{4.5}\right) + 5.5 + 2.83\left(\frac{5.67}{9}\right)\right]\mathrm{ft} = 14.56\,\mathrm{ft}$$

Percent full height sheathing: $\% := \left(\frac{10\cdot\mathrm{ft}}{10\cdot\mathrm{ft}}\right)\cdot100$ $\% = 100$

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

$vaa = 330.49\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$ $\frac{vaa}{C_o} = 330.49\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$

$E_{aa} = 226.37\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$ $\frac{E_{aa}}{C_o} = 226.37\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$

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

Dead Load Resisting Overturning: $L_{aa} := 2.83\cdot\mathrm{ft}$ Plate Height: $Pt := 9\cdot\mathrm{ft}$

$W_{aa} := (15\cdot\mathrm{psf})\cdot2\cdot\mathrm{ft} + (10\cdot\mathrm{psf})\cdot Pt + (10\mathrm{psf})\cdot0\mathrm{ft}$

$DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 169.8\,\mathrm{lb}$

Chord Force:

$CFaa_w := \frac{vaa\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 2974.42\,\mathrm{lb}$

$CFaa_s := \frac{E_{aa}\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 2037.29\,\mathrm{lb}$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = 2872.54\,\mathrm{lb}$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 1957.5\,\mathrm{lb}$

Simpson MSTC40

Note: T.O.W. to bottom of header is 16" at Main Floor below
(40" strap - 12" floor cavity)/2 = 14"

Dead Load Resisting Overturning: $L_{aa} := 17 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{aa1} := (15 \cdot psf) \cdot 19 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRaa1 := \dfrac{W_{aa1} \cdot L_{aa}}{2}$ $DLRaa = 169.8\,lb$

Chord Force:

$CFaa_{w1} := \dfrac{vaa \cdot 7.67ft \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_{w1} = 1341.99\,lb$

$CFaa_{s1} := \dfrac{E_{aa} \cdot 7.67ft \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_{s1} = 919.18\,lb$

Holdown Force:

$HDFaa_{w1} := CFaa_{w1} - 0.6 \cdot DLRaa1 = -570.51\,lb$

$HDFaa_{s1} := CFaa_{s1} - \left(0.6 - 0.14S_{DS}\right)DLRaa1 = -578.61\,lb$

No Holdown Required, use CSHP20 horizontal straps above & below openings

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

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

$B_p := \dfrac{\left(Z_N \cdot C_D \cdot C_o\right)}{vaa} = 0.49\,ft$ $\dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_{aa}} = 0.72\,ft$

16d @ 6" o.c.

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

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

$As := \dfrac{\left(Z_B \cdot C_o\right)}{vaa} = 4.16\,ft$ $\dfrac{\left(Z_B \cdot C_o\right)}{E_{aa}} = 6.08\,ft$

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

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WALL BB:

Story Shear due to Wind: $V_{3W} = 19102.36\,lb$

Story Shear due to Seismic: $F_1 = 9417.9\,lb$

Bldg Width in direction of Load: $L_t := 40.5 \cdot ft$

Distance between shear walls: $L_1 := 40.5 \cdot ft$

Shear Wall Length:

$Lbb_w := (2 \cdot 3.5 + 3.25 + 4.33 + 2.83)ft = 17.41\,ft$

$$Lbb_s := \left[2 \cdot 3.5\left(\frac{7}{9}\right) + 3.25\left(\frac{6.5}{9}\right) + 4.33\left(\frac{8.67}{9}\right) + 2.83\left(\frac{5.67}{9}\right)\right]ft = 13.75\,ft$$

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

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

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

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

$vbb = 329.16\,ft^{-1} \cdot lb$ $\qquad \frac{vbb}{C_o} = 329.16\,ft^{-1} \cdot lb$

$E_{bb} = 239.8\,ft^{-1} \cdot lb$ $\qquad \frac{E_{bb}}{C_o} = 239.8\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{bb} := 2.83 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $\qquad DLRbb = 169.8\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\qquad CFbb_w = 2962.46\,lb$

$CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\qquad CFbb_s = 2158.22\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 2860.58\,lb$

$HDFbb_s := CFbb_s - \left(0.6 - 0.14S_{DS}\right) \cdot DLRbb = 2078.43\,lb$

Simpson MSTC40

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

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

$B_R := \frac{\left(C_D \cdot Z_N \cdot C_o\right)}{vbb} = 0.5\,ft$ $\qquad \frac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_{bb}} = 0.68\,ft$

16d @ 6" o.c.

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

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

$AS := \frac{\left(Z_B \cdot C_o\right)}{vbb} = 4.18\,ft$ $\qquad \frac{\left(Z_B \cdot C_o\right)}{E_{bb}} = 5.74\,ft$

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

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WALL CC:

Story Shear due to Wind: $V_{1W} = 16900.81\,\mathrm{lb}$ Story Shear due to Seismic: $F_1 = 9417.9\,\mathrm{lb}$

Bldg Width in direction of Load: $L_t := 47.5\cdot ft$ Distance between shear walls: $L_1 := 47.5\cdot ft$

Shear Wall Length: $Lcc_w := (10.17 + 6.67)ft = 16.84\,ft$ $Lcc_s := (10.17 + 6.67)ft = 16.84\,ft$

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

$vcc = 301.08\,ft^{-1}\cdot lb$ $\frac{vcc}{C_o} = 301.08\,ft^{-1}\cdot lb$ $E_{cc} = 195.74\,ft^{-1}\cdot lb$ $\frac{E_{cc}}{C_o} = 195.74\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 6.67\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{cc} := (15\cdot psf)\cdot 10\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRcc := \frac{W_{cc}\cdot L_{cc}}{2}$ $DLRcc = 800.4\,lb$

Chord Force:

$CFcc_w := \frac{vcc\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_w = 2709.75\,lb$ $CFcc_s := \frac{E_{cc}\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_s = 1761.66\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 2229.51\,lb$ $HDFcc_s := CFcc_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRcc = 1385.56\,lb$

Simpson MSTC40 to wall below, or ST6224 Direct to beam

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{vcc} = 0.54\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{cc}} = 0.83\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vcc} = 4.57\,ft$ $\frac{(Z_B\cdot C_o)}{E_{cc}} = 7.03\,ft$

5/8" A.B. @ 54" o.c.

WALL DD:

Story Shear due to Wind: $V_{1W} = 16900.81\,lb$ Story Shear due to Seismic: $F_1 = 9417.9\,lb$

Bldg Width in direction of Load: $L_t := 47.5\cdot ft$ Distance between shear walls: $L_1 := 47.5\cdot ft$

Shear Wall Length: $Ldd_w := (2\cdot 7.33)ft = 14.66\,ft$ $Ldd_s := (2\cdot 7.33)ft = 14.66\,ft$

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

$vdd = 345.86\,ft^{-1}\cdot lb$ $\dfrac{vdd}{C_o} = 345.86\,ft^{-1}\cdot lb$

$E_{dd} = 224.85\,ft^{-1}\cdot lb$ $\dfrac{E_{dd}}{C_o} = 224.85\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 7.33\cdot ft$ Plate Height: $Pt := 9\cdot ft$

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

$DLRdd := \dfrac{W_{dd}\cdot L_{dd}}{2}$ $DLRdd = 769.65\,lb$

Chord Force:

$CFdd_w := \dfrac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_w = 3112.7\,lb$

$CFdd_s := \dfrac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_s = 2023.63\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 2650.91\,lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 1661.98\,lb$

Simpson MSTC40

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

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

$B_P := \dfrac{(C_D\cdot Z_N\cdot C_o)}{vdd} = 0.47\,ft$ $\dfrac{(C_D\cdot Z_N\cdot C_o)}{E_{dd}} = 0.73\,ft$

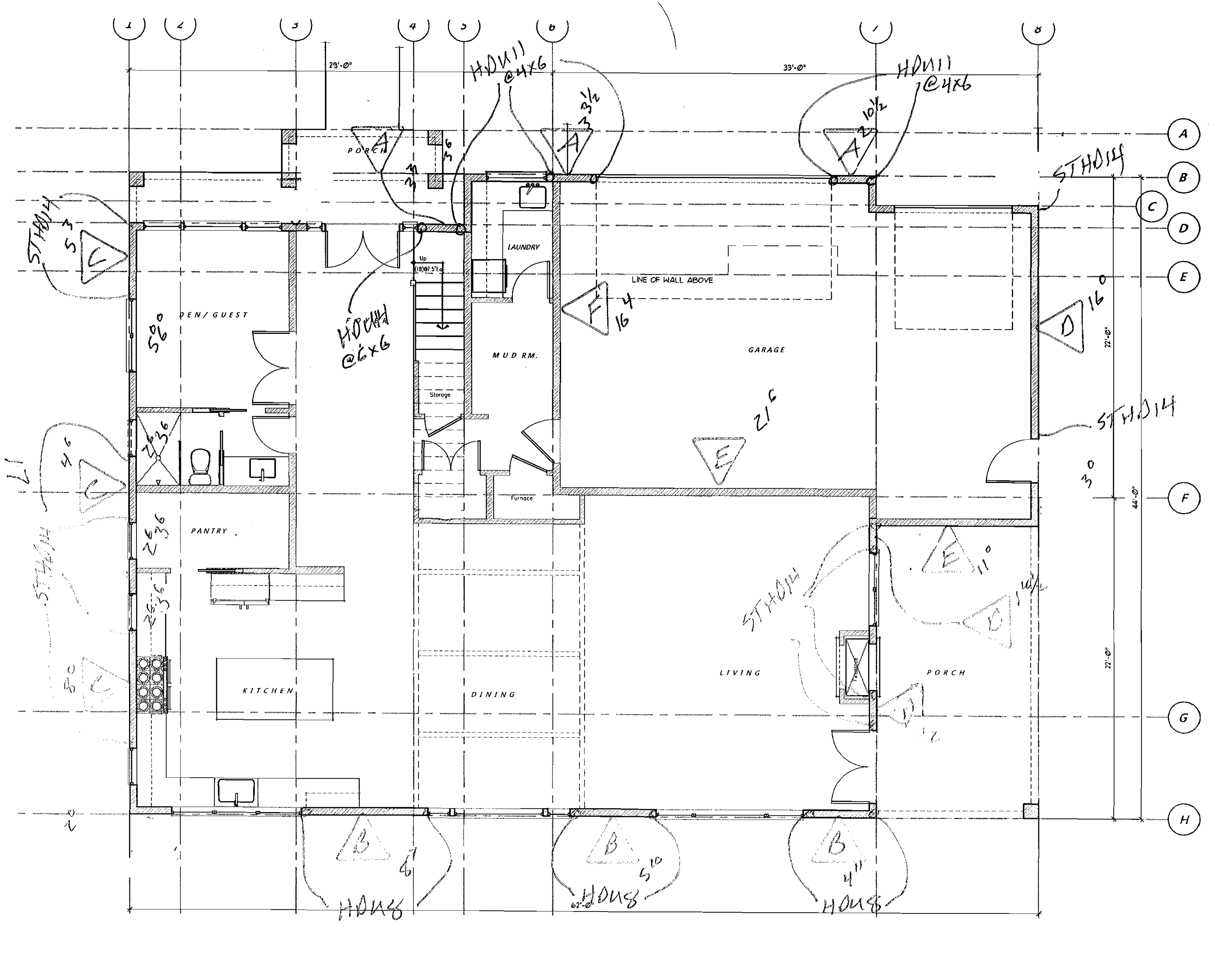
16d @ 6" o.c.

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

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

$As := \dfrac{(Z_B\cdot C_o)}{vdd} = 3.98\,ft$ $\dfrac{(Z_B\cdot C_o)}{E_{dd}} = 6.12\,ft$

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



29'-0"
33'-0"
22'-0"
44'-0"
62'-0"
PORCH
DEN/GUEST
LAUNDRY
MUD RM.
Storage
Furnace
Up
LINE OF WALL ABOVE
GARAGE
PANTRY
KITCHEN
DINING
LIVING
HDU11 @4x6
HDU4 @6x6
STHD14
HDU8

WALL A:

Story Shear due to Wind: $V_{4W} = 19817.93\,\text{lb}$ Story Shear due to Seismic: $F_2 = 6819.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 44\cdot\text{ft}$ Distance between shear walls: $L_1 := 22\cdot\text{ft}$

Shear Wall Length:

$La_w := (3.25 + 3.29 + 2.875)\text{ft} = 9.41\,\text{ft}$

$$La_s := \left[3.25\left(\frac{6.5}{10}\right) + 3.29\left(\frac{6.58}{9}\right) + 2.875\left(\frac{5.75}{10}\right)\right]\text{ft} = 6.17\,\text{ft}$$

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

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

Wind Force: $$va := \frac{vaa\cdot Laa_w + \left(\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}\right)}{La_w}$$

Seismic Force: $\rho := 1.0$ $$E_a := \frac{E_{aa}\cdot Laa_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{La_s}$$

$va = 924.42\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{va}{C_o} = 924.42\,\text{ft}^{-1}\cdot\text{lb}$ $\quad E_a = 727.55\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{E_a}{C_o} = 727.55\,\text{ft}^{-1}\cdot\text{lb}$

P1-2: 7/16" Sheathing w/ 10d nails @ 2" O.C.
Wind Capacity = 1077 plf
Seismic Capacity = 770 plf

Dead Load Resisting Overturning: $L_a := 2.875\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_a := (15\cdot\text{psf})\cdot 2\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$ $\quad DLRa := \frac{W_a\cdot L_a}{2}$ $\quad DLRa = 186.88\,\text{lb}$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $\quad CFa_w = 9244.18\,\text{lb}$ $\quad CFa_w + CFaa_w = 12218.6\,\text{lb}$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $\quad CFa_s = 7275.48\,\text{lb}$ $\quad CFa_s + CFaa_s = 9312.76\,\text{lb}$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 9132.06\,\text{lb}$ $\quad HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 7187.67\,\text{lb}$

Simpson HDU11 at 4x6 post w/ SB1x30 at corner or midwall & PAB8 at end wall w/ 8" embed in 24" wide footing

$HDFa_w + HDFaa_w = 12004.59\,\text{lb}$ $\quad HDFa_s + HDFaa_s = 9145.16\,\text{lb}$

Simpson HDU14 at 6x6 post w/ SB1x30 at midwall

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.18\,\text{ft}$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 0.22\,\text{ft}$

16d @ 2" o.c. (STAGGER)

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

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

$As := \frac{(Z_B\cdot C_o)}{va} = 1.49\,\text{ft}$ $\quad \frac{(Z_B\cdot C_o)}{E_a} = 1.89\,\text{ft}$

5/8" A.B. @ 18" o.c.

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WALL B:

Story Shear due to Wind: $V_{4W} = 19817.93\,lb$ Story Shear due to Seismic: $F_2 = 6819.52\,lb$

Bldg Width in direction of Load: $L_t := 44\cdot ft$ Distance between shear walls: $L_1 := 22\cdot ft$

Shear Wall Length:

$Lb_w := (8.58 + 5.83 + 4.92)ft = 19.33\,ft$ $Lb_s := \left[8.58 + 5.83 + 4.92\left(\frac{9.83}{10}\right)\right]ft = 19.25\,ft$

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

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

Wind Force: $vb := \dfrac{vbb\cdot Lbb_w + \left(\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lb_w}$ Seismic Force: $\rho := 1.0$ $E_b := \dfrac{E_{bb}\cdot Lbb_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Lb_s}$

$vb = 450.25\,ft^{-1}\cdot lb$ $\frac{vb}{C_o} = 450.25\,ft^{-1}\cdot lb$ $E_b = 233.27\,ft^{-1}\cdot lb$ $\frac{E_b}{C_o} = 233.27\,ft^{-1}\cdot lb$

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

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

$W_b := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$ $DLRb := \frac{W_b\cdot L_b}{2}$ $DLRb = 270.6\,lb$

Chord Force:

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_w = 4502.53\,lb$ $CFb_w + CFbb_w = 7464.99\,lb$

$CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_s = 2332.74\,lb$ $CFb_s + CFbb_s = 4490.96\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 4340.17\,lb$ $HDFb_w + HDFbb_w = 7200.75\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14S_{DS})\cdot DLRb = 2205.59\,lb$ $HDFb_s + HDFbb_s = 4284.02\,lb$

Simpson HDU8 at 4x6 post (min) w/ SB7/8x24 anchor

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

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

$B_N := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 0.36\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.7\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vb} = 3.06\,ft$ $\frac{(Z_B\cdot C_o)}{E_b} = 5.9\,ft$

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

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WALL C:

Story Shear due to Wind: $V_{2W} = 26764.05\,lb$ Story Shear due to Seismic: $F_2 = 6819.52\,lb$

Bldg Width in direction of Load: $L_t := 62 \cdot ft$ Distance between shear walls: $L_1 := 29 \cdot ft$

Shear Wall Length: $Lc_w := (8.0 + 5.25 + 4.5)ft = 17.75\,ft$ $Lc_s := \left[8.0 + 5.25 + 4.5\left(\frac{9}{10}\right)\right]ft = 17.3\,ft$

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

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

$vc = 497.23\,ft^{-1} \cdot lb$ $\frac{vc}{C_o} = 497.23\,ft^{-1} \cdot lb$ $E_c = 255.07\,ft^{-1} \cdot lb$ $\frac{E_c}{C_o} = 255.07\,ft^{-1} \cdot lb$

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

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

$W_c := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 2ft$ $DLRc := \frac{W_c \cdot L_c}{2}$ $DLRc = 270\,lb$

Chord Force:

$CFc_w := \frac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $CFc_w = 4972.31\,lb$ $CFc_s := \frac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $CFc_s = 2550.69\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 4810.31\,lb$ $HDFc_s := CFc_s - (0.6 - 0.14S_{DS}) \cdot DLRc = 2423.82\,lb$

Simpson STHD14

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vc} = 0.33\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 0.64\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vc} = 2.77\,ft$ $\frac{(Z_B \cdot C_o)}{E_c} = 5.39\,ft$

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

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PROJECT : **3404 72nd Place SE**

WALL D:

Story Shear due to Wind: $V_{2W} = 26764.05\,\text{lb}$

Story Shear due to Seismic: $F_2 = 6819.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 62\cdot\text{ft}$

Distance between shear walls: $L_1 := 33\cdot\text{ft}$

Shear Wall Length: $Ld_w := (16 + 1.875 + 2)\text{ft} = 19.88\,\text{ft}$ $\quad Ld_s := (16 + 1.875 + 2)\text{ft} = 19.88\,\text{ft}$

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

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

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

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

$vd = 470.13\,\text{ft}^{-1}\cdot\text{lb} \qquad \dfrac{vd}{C_o} = 470.13\,\text{ft}^{-1}\cdot\text{lb}$

$E_d = 229.77\,\text{ft}^{-1}\cdot\text{lb} \qquad \dfrac{E_d}{C_o} = 229.77\,\text{ft}^{-1}\cdot\text{lb}$

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

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

Restraint Panel Height = 10ft Maximum

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

Allowable Shear per Panel = 1031 lbs Seismic & 1444 lbs Wind

Shear per Panel: $V_s := (2\text{ft}\cdot E_d) = 459.54\,\text{lb}$ O.K.

$V_w := (2\text{ft}\cdot vd) = 940.26\,\text{lb}$ O.K.

Dead Load Resisting Overturning: $L_d := 16\cdot\text{ft}$ $\quad$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_d := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 6.5\text{ft}$

$DLRd := \dfrac{W_d\cdot L_d}{2} \qquad DLRd = 1320\,\text{lb}$

Chord Force:

$CFd_w := \dfrac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d} \qquad CFd_w = 4701.31\,\text{lb}$

$CFd_s := \dfrac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d} \qquad CFd_s = 2297.7\,\text{lb}$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 3909.31\,\text{lb}$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 1677.44\,\text{lb}$

SImpson STHD14

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

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

$B_R := \dfrac{(C_D\cdot Z_N\cdot C_o)}{vd} = 0.35\,\text{ft} \qquad \dfrac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 0.71\,\text{ft}$

16d @ 4" o.c.

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

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

$A_S := \dfrac{(Z_B\cdot C_o)}{vd} = 2.93\,\text{ft} \qquad \dfrac{(Z_B\cdot C_o)}{E_d} = 5.99\,\text{ft}$

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

WALL E:

Story Shear due to Wind: $V_{4W} = 19817.93\,lb$

Story Shear due to Seismic: $F_2 = 6819.52\,lb$

Bldg Width in direction of Load: $L_t := 44 \cdot ft$

Distance between shear walls: $L_1 := 22 \cdot ft$ $L_2 := 22ft$

Shear Wall Length: $Le_w := (21.5 + 11)ft = 32.5\,ft$ $Le_s := (21.5 + 11)ft = 32.5\,ft$

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: $ve := \frac{\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Le_w}$

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

$ve = 182.93\,ft^{-1} \cdot lb$ $\frac{ve}{C_o} = 182.93\,ft^{-1} \cdot lb$

$E_e = 73.44\,ft^{-1} \cdot lb$ $\frac{E_e}{C_o} = 73.44\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_e := 11 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_e := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRe := \frac{W_e \cdot L_e}{2}$ $DLRe = 715\,lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_w = 1829.35\,lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_s = 734.41\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6DLRe = 1400.35\,lb$

$HDFe_s := CFe_s - (0.6 - 0.14S_{DS}) \cdot DLRe = 398.44\,lb$

SImpson LSTHD8 or HDU2 w/ SSTB16 anchor

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{ve} = 0.89\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 2.22\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{ve} = 7.52\,ft$ $\frac{(Z_B \cdot C_o)}{E_e} = 18.74\,ft$

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

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WALL F:

Story Shear due to Wind: $V_{2W} = 26764.05\,lb$

Story Shear due to Seismic: $F_2 = 6819.52\,lb$

Bldg Width in direction of Load: $L_t := 62 \cdot ft$

Distance between shear walls: $L_1 := 29 \cdot ft$ $L_2 := 33ft$

Shear Wall Length: $Lf_w := (16.33)ft = 16.33\,ft$ $Lf_s := (16.33)ft = 16.33\,ft$

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

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

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

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

$vf = 491.68\,ft^{-1} \cdot lb$ $\frac{vf}{C_o} = 491.68\,ft^{-1} \cdot lb$

$E_f = 146.16\,ft^{-1} \cdot lb$ $\frac{E_f}{C_o} = 146.16\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_f := 16.33 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 14ft$

$DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 1959.6\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 4916.85\,lb$

$CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 1461.62\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 3741.09\,lb$

$HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 540.82\,lb$

Simpson HDU4 w/ SSTB20 anchor

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 0.33\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 1.12\,ft$

16d @ 4" o.c.

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

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

$AS := \frac{(Z_B \cdot C_o)}{vf} = 2.8\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 9.41\,ft$

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

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa \cdot \frac{Laa_w}{47.5ft} = 120.65\,ft^{-1} \cdot lb \qquad E_{aa} \cdot \frac{Laa_s}{47.5ft} = 69.4\,ft^{-1} \cdot lb$$

Wall Lines BB:

$$vbb \cdot \frac{Lbb_w}{47.5ft} = 120.65\,ft^{-1} \cdot lb \qquad E_{bb} \cdot \frac{Lbb_s}{47.5ft} = 69.4\,ft^{-1} \cdot lb$$

Wall Lines CC:

$$vcc \cdot \frac{Lcc_w}{40.5ft} = 125.19\,ft^{-1} \cdot lb \qquad E_{cc} \cdot \frac{Lcc_s}{40.5ft} = 81.39\,ft^{-1} \cdot lb$$

Wall Lines DD:

$$vdd \cdot \frac{Ldd_w}{37ft} = 137.03\,ft^{-1} \cdot lb \qquad E_{dd} \cdot \frac{Ldd_s}{37ft} = 89.09\,ft^{-1} \cdot lb$$

Assume DF Floor Framing, 15/32"" Sheathing w/ 8d (0.131" x 2.5") nails, 6" o.c Edge nailing

Unblocked Diapraghm Case 1 Wind Capacity = 335 plf & Seismic Capacity = 240 plf

Unblocked Diapraghm Case 2-6 Wind Capacity = 253 plf & Seismic Capacity = 180 plf

Wall Lines A:

$$\frac{va \cdot La_w - vaa \cdot Laa_w}{51ft} = 58.29\,ft^{-1} \cdot lb \qquad \frac{E_a \cdot La_s - E_{aa} \cdot Laa_s}{51ft} = 23.4\,ft^{-1} \cdot lb \qquad \frac{va \cdot La_w}{51ft} = 170.65\,ft^{-1} \cdot lb \qquad \frac{E_a \cdot La_s}{51ft} = 88.03\,ft^{-1} \cdot lb$$

Wall Lines B:

$$\frac{vb \cdot Lb_w - vbb \cdot Lbb_w}{51ft} = 58.29\,ft^{-1} \cdot lb \qquad \frac{E_b \cdot Lb_s - E_{bb} \cdot Lbb_s}{51ft} = 23.4\,ft^{-1} \cdot lb \qquad \frac{vb \cdot Lb_w}{51ft} = 170.65\,ft^{-1} \cdot lb \qquad \frac{E_b \cdot Lb_s}{51ft} = 88.03\,ft^{-1} \cdot lb$$

Wall Lines C:

$$\frac{vc \cdot Lc_w - vcc \cdot Lcc_w}{41ft} = 91.6\,ft^{-1} \cdot lb \qquad \frac{E_c \cdot Lc_s - E_{cc} \cdot Lcc_s}{41ft} = 27.23\,ft^{-1} \cdot lb \qquad \frac{vc \cdot Lc_w}{41ft} = 215.26\,ft^{-1} \cdot lb \qquad \frac{E_c \cdot Lc_s}{41ft} = 107.63\,ft^{-1} \cdot lb$$

Wall Lines D:

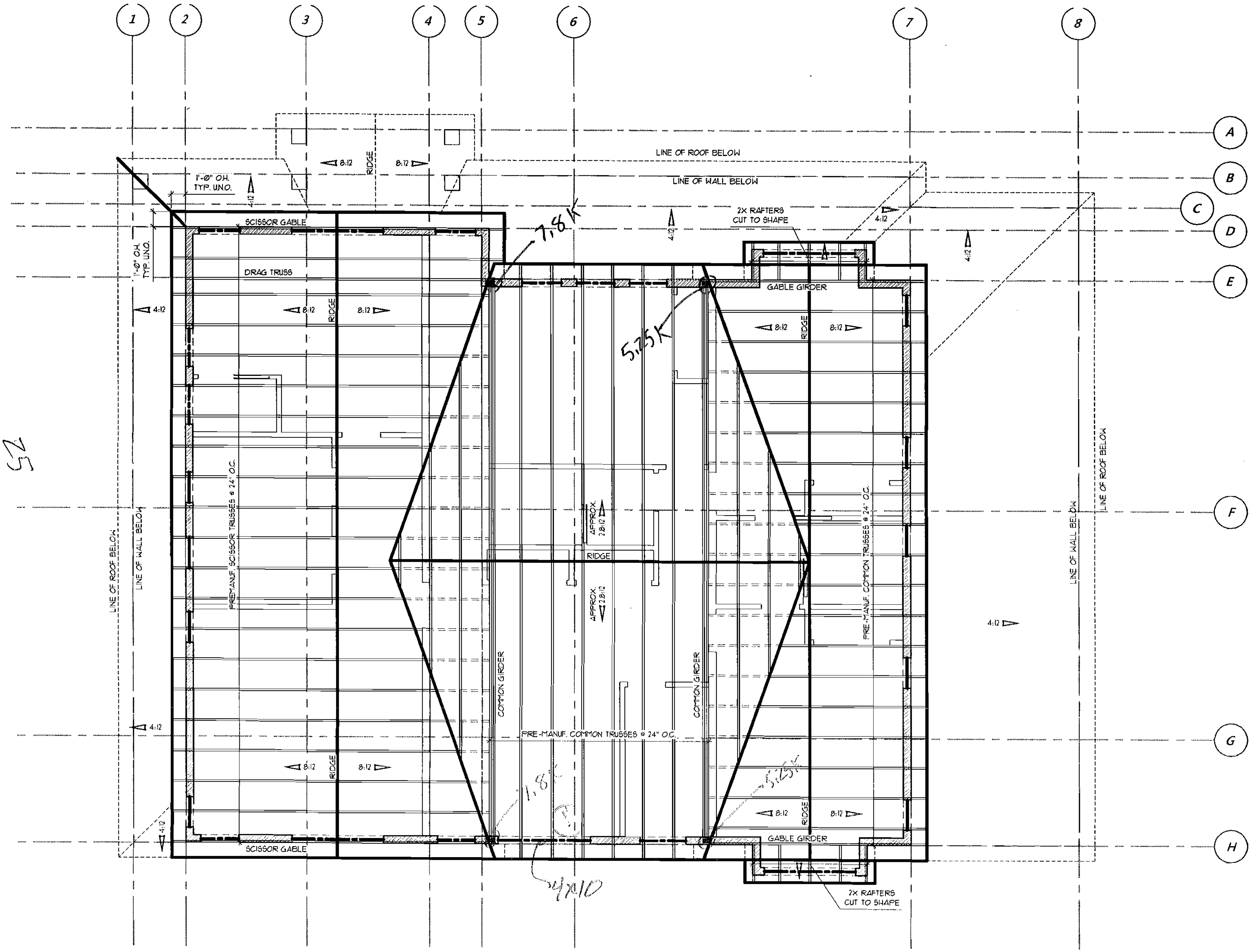
$$\frac{vd \cdot Ld_w - vdd \cdot Ldd_w}{42ft} = 101.75\,ft^{-1} \cdot lb \qquad \frac{E_d \cdot Ld_s - E_{dd} \cdot Ldd_s}{42ft} = 30.25\,ft^{-1} \cdot lb \qquad \frac{vd \cdot Ld_w}{42ft} = 222.47\,ft^{-1} \cdot lb \qquad \frac{E_d \cdot Ld_s}{42ft} = 108.73\,ft^{-1} \cdot lb$$

Wall Lines E:

$$ve \cdot \frac{Le_w}{41ft} = 145.01\,ft^{-1} \cdot lb \qquad E_e \cdot \frac{Le_s}{41ft} = 58.22\,ft^{-1} \cdot lb$$

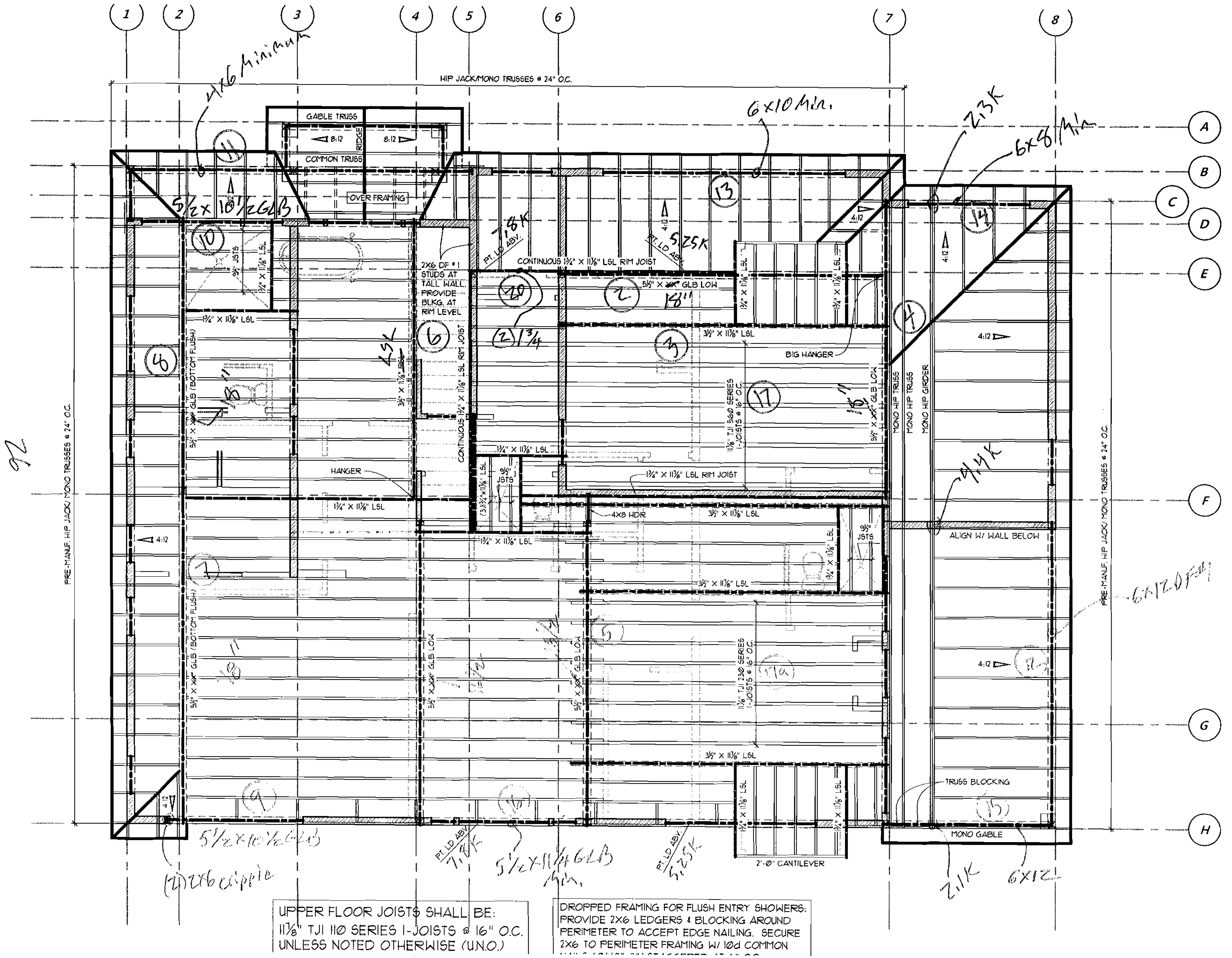
Wall Lines F:

$$vf \cdot \frac{Lf_w}{44ft} = 182.48\,ft^{-1} \cdot lb \qquad E_f \cdot \frac{Lf_s}{44ft} = 54.25\,ft^{-1} \cdot lb$$

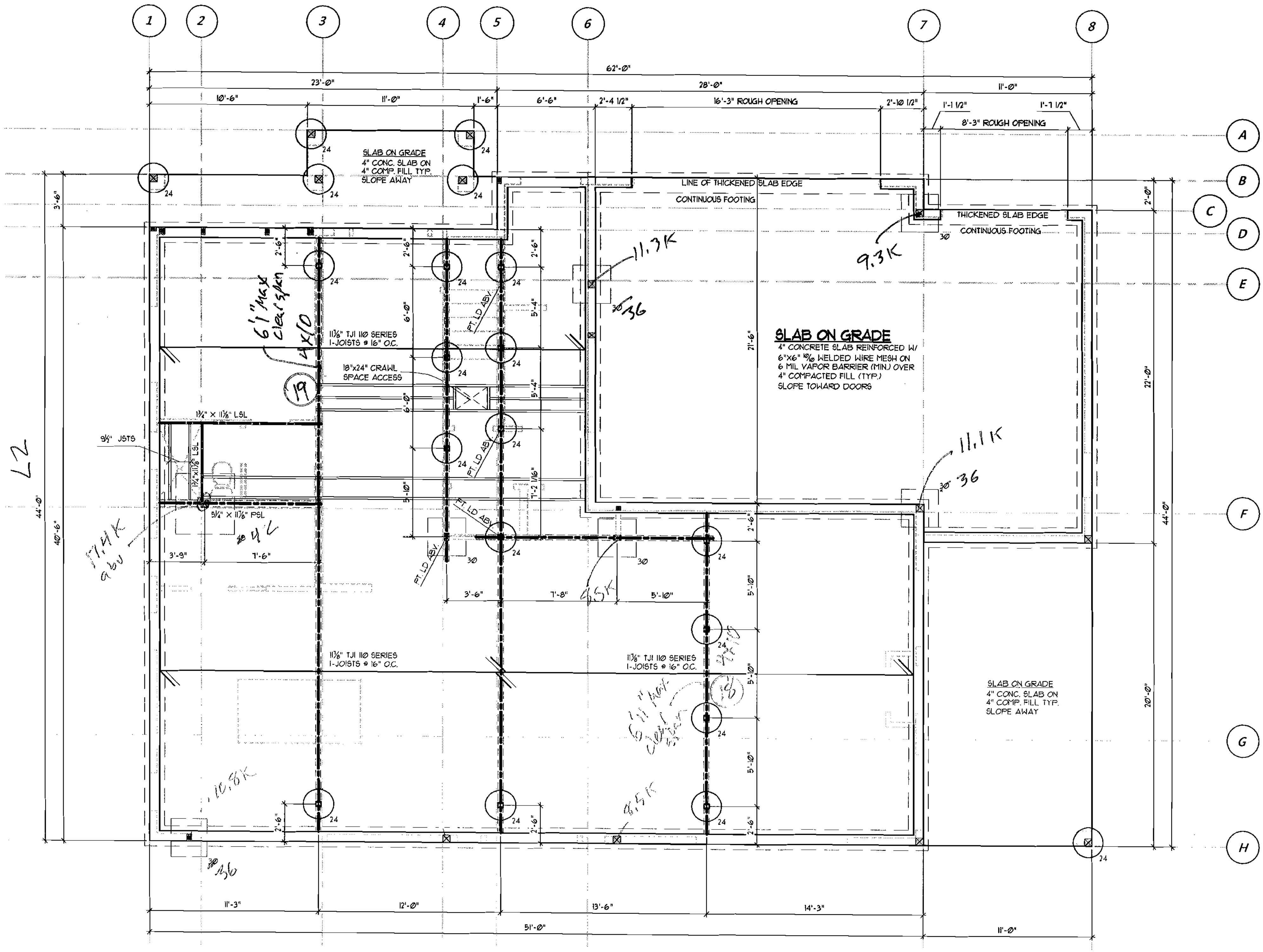


1
2
3
4
5
6
7
8
A
B
C
D
E
F
G
H
LINE OF ROOF BELOW
LINE OF WALL BELOW
1'-0" O.H. TYP. UNO.
SCISSOR GABLE
DRAG TRUSS
RIDGE
8:12
4:12
PREMANUF. SCISSOR TRUSSES @ 24" O.C.
7.8K
5.25K
2X RAFTERS CUT TO SHAPE
GABLE GIRDER
COMMON GIRDER
APPROX. 2.8:12
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
4X10

Z5



HIP JACK/MONO TRUSSES @ 24" O.C.
PRE-MANUF. HIP JACK/ MONO TRUSSES @ 24" O.C.
GABLE TRUSS
COMMON TRUSS
RIDGE
OVER FRAMING
2X6 DF #1 STUDS AT TALL WALL. PROVIDE BLKG. AT RIM LEVEL
CONTINUOUS 1¾" X 11⅞" LSL RIM JOIST
5½" X XX" GLB LOW
3½" X 11⅞" LSL
1¾" X 11⅞" LSL
BIG HANGER
HANGER
11⅞" TJI 560 SERIES I-JOISTS @ 16" O.C.
11⅞" TJI 230 SERIES I-JOISTS @ 16" O.C.
1¾" X 11⅞" LSL RIM JOIST
4X8 HDR
9½" JSTS
5½" X XX" GLB (BOTTOM FLUSH)
MONO HIP TRUSS
MONO HIP GIRDER
ALIGN W/ WALL BELOW
TRUSS BLOCKING
MONO GABLE
2'-0" CANTILEVER
PT. LD ABV.
4:12
8:12
4x6 Minimum
6x10 Min.
2.3K
6x8 Min
5½x10½GLB
7.8K
5.25K
18"
(2)1¾
4.4K
6x12 DF#1
5½x10½GLB
(2)2x6 cripple
7.8K
5½x11¼GLB Min.
5.25K
2.1K
6x12
UPPER FLOOR JOISTS SHALL BE:
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
DROPPED FRAMING FOR FLUSH ENTRY SHOWERS:
PROVIDE 2X6 LEDGERS & BLOCKING AROUND
PERIMETER TO ACCEPT EDGE NAILING. SECURE
2X6 TO PERIMETER FRAMING W/ 10d COMMON
26


SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED FILL (TYP.)
SLOPE TOWARD DOORS
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
LINE OF THICKENED SLAB EDGE
CONTINUOUS FOOTING
THICKENED SLAB EDGE
CONTINUOUS FOOTING
16'-3" ROUGH OPENING
8'-3" ROUGH OPENING
11 7/8" TJI 110 SERIES
I-JOISTS @ 16" O.C.
18"x24" CRAWL
SPACE ACCESS
1 3/4" X 11 7/8" LSL
5 1/4" X 11 7/8" PSL
9 1/2" JSTS
PT. LD ABV.
62'-0"
23'-0"
28'-0"
11'-0"
51'-0"
44'-0"
40'-6"
9.3K
11.3K
11.1K
6.5K
9.5K
10.8K
17.4K
6'1" max
clear span
4x10
19
L2

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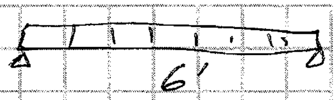
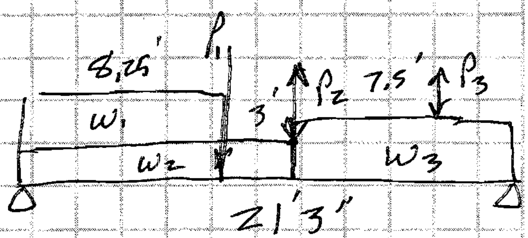
(1) $w_D = 15psf\,(39'/2) = 292.5plf$
$w_S = 25psf\,(39'/2) = 487.5plf$



4x10

(2) $w_{D_1} = 292.5plf + 12psf\,(9') = 400.5plf$
$w_{S_1} = 487.5plf$

$w_{D2} = 15psf\,(7'/2 + 1') = 67.5plf$
$w_{L2} = 40psf\,(1') = 40plf$
$w_{S2} = 25psf\,(7'/2) = 87.5plf$



$P_1 = 2125^{\#}DL + 3125^{\#}SL$ from Girder

5½x18 GLB

$w_{D3} = 15psf\,(8'/2 + 5'/2 + 4'/2 + 1') + 12psf\,(9') = 250.5plf$
$w_{L3} = 40psf\,(8'/2) = 160plf$
$w_{S3} = 25psf\,(5'/2 + 4'/2 + 1') = 137.5plf$

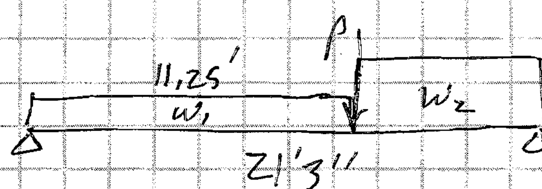
$P_2 = \pm 1350^{\#}WL \pm 925^{\#}EL$ w/ $\Omega = 3.0$

$P_3 = \pm 2975^{\#}WL \pm 2050^{\#}EL$ w/ $\Omega = 3.0$

(3) $w_{D_1} = 15psf\,(1.33') = 20plf$
$w_{L_1} = 40psf\,(1.33' = 53.3plf$

$w_{D2} = 15psf\,(5'/2) = 37.5plf$
$w_{L2} = 40psf\,(5'/2) = 100plf$

$P = 150^{\#}DL + 400^{\#}LL$



3½x11⅞ LSL

FOR Lot 3 DATE 7-6-20
JOB BY MM

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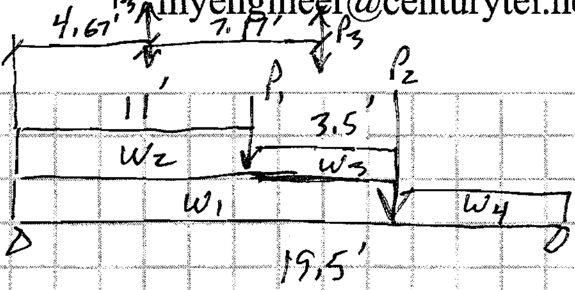


5½x18 GLB

(4) $w_{D1} = 15psf(15.5'/2) + 12psf(9') = 224.25plf$
$w_{S1} = 25psf(15.5'/2) = 193.75plf$

$w_{D2} = 15psf(21.25'/2) = 159.38plf$
$w_{L2} = 40psf(21.25'/2) = 425plf$

$P_1 = 430^{\#}DL + 1140^{\#}LL$ from (3)

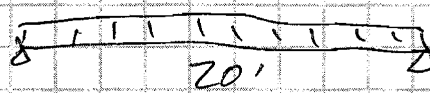
$w_{D3} = 15psf(1') = 15plf$
$w_{L3} = 40psf(1') = 40plf$

$P_2 = 3585^{\#}DL + 1345^{\#}LL + 3310^{\#}SL \pm 1910^{\#}WL \pm 1319^{\#}EL$ w/ $\Omega = 3.0$ from (2)

$w_{D4} = 15psf(4'/2) = 30plf$
$w_{S4} = 25psf(4'/2) = 50plf$

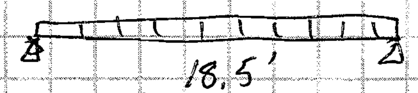
$P_3 = \pm 3125^{\#}WL \pm 2025^{\#}EL$ w/ $\Omega = 3.0$

(5) $w_D = 15psf(31'/2) = 232.5plf$
$w_L = 40psf(31'/2) = 620plf$



5½x16½ GLB

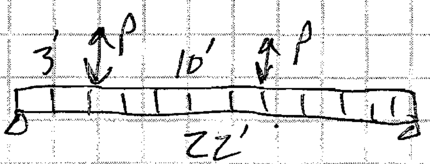
(6) $w_D = 15psf(8'/2) = 60plf$
$w_L = 40psf(8'/2) = 160plf$



3½x11⅞ LSL

(7) $w_D = 15psf(22'/2 + 3.5'/2 + 16'/2) + 12psf(9') = 419.25plf$
$w_L = 40psf(16'/2) = 320plf$
$w_S = 25psf(22'/2 + 3.5'/2) = 318.75plf$

$P = \pm 2725^{\#}WL \pm 1775^{\#}EL$ w/ $\Omega = 3.0$



5½x18 GLB

FOR Lot 3
JOB

DATE 7-6-20
BY MM

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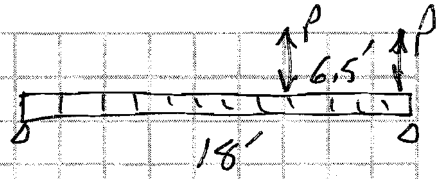
(8) $w_D = 15psf(22'/2 + 3.5'/2 + 8'/2) + 12psf(9') = 359.25plf$
$w_L = 40psf(8'/2) = 160plf$
$w_S = 25psf(22'/2 + 3.5'/2) = 318.75plf$

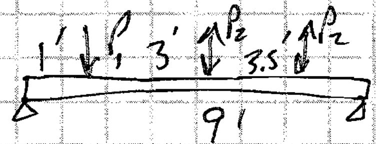
$P = \pm 2725^{\#} WL \pm 1775^{\#} EL$ w/ $\Omega = 3.0$



5½ x 15 GLB Minimum

(9) $w_D = 15psf(2' + 1') + 12psf(9') = 153plf$
$w_L = 40plf$
$w_S = 25psf(2') = 50plf$

$P_1 = 4620^{\#}DL + 3530^{\#}LL + 3520^{\#}SL \pm 1250^{\#}WL \pm 820^{\#}EL$ from (7)
$P_2 = \pm 2975^{\#} WL \pm 2175^{\#} EL$ w/ $\Omega = 3.0$



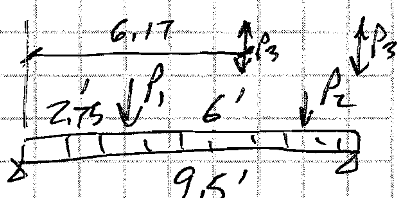
5½x10½ GLB

(10) $w_D = 153plf + 15psf(3.5'/2) = 179.25plf$
$w_L = 40plf$
$w_S = 50plf + 25psf(3.5'/2) = 93.75plf$

$P_1 = 3230^{\#}DL + 1440^{\#}LL + 2870^{\#}SL \pm 990^{\#}WL \pm 650$ from (8)

$P_2 = 180^{\#}DL + 480^{\#}LL$ from shower

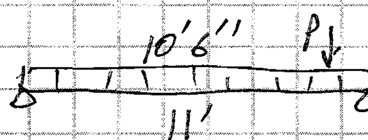
$P_3 = \pm 2975^{\#}WL \pm 2050^{\#}EL$ w/ $\Omega = 3.0$



5½x9 GLB Minimum

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

$P = 150^{\#}DL + 250^{\#}SL$ from Porch Roof



4x6 Minimum

FOR Lot 3
JOB

DATE 7-6-20
BY [signature]

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Gig Harbor, WA 98335
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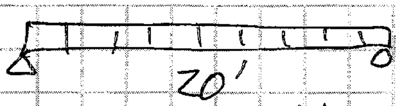
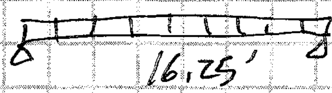
myengineer@centurytel.net

(12) $w_D = 15psf\left(\frac{10'}{2}\right) = 75plf$
$w_S = 25psf\left(\frac{10'}{2}\right) = 125plf$



6x12 DF #1

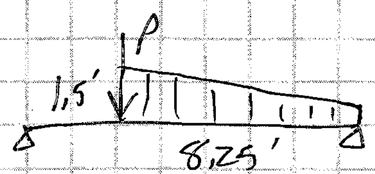
(13) $w_D = 15psf\left(\frac{9'}{2}\right) = 67.5plf$
$w_S = 25psf\left(\frac{9'}{2}\right) = 112.5plf$



6x10 Minimum

(14) $w_D = 75plf$
$w_S = 125plf$

$P = 950^{\#}DL + 1400^{\#}SL$ from Girder

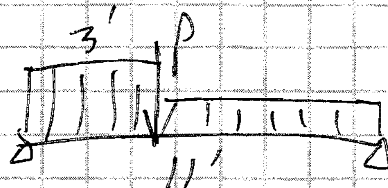


6x8 Min

(15) $w_{D1} = 15psf\left(\frac{22'}{2}\right) = 165plf$
$w_{S1} = 25psf\left(\frac{22'}{2}\right) = 275plf$

$P = 850^{\#}DL + 1250^{\#}SL$ from Girder

$w_{D2} = 30plf$
$w_{S2} = 50plf$

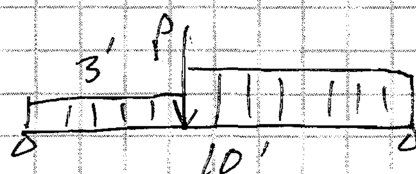


6x10 Min

(16) $w_{D1} = 30plf + 12psf(9') = 138plf$
$w_{L1} = 40plf$
$w_{S1} = 50plf$

$P = 3160^{\#}DL + 4650^{\#}SL$ from girder

$w_{D2} = 15psf\left(\frac{39'}{2} + 1\right) + 12psf(9') = 415.5plf$
$w_{S2} = 25psf\left(\frac{39'}{2}\right) = 487.5plf$
$w_{L2} = 40plf$



5½x11¼ GLB
Minimum
OR W8x15 Steel
($F_y = 50ksi$)

FOR Lot 3
JOB

DATE 7-6-20
BY MM

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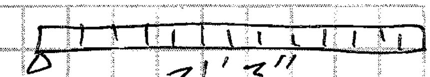
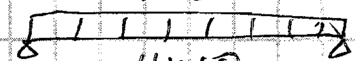
(17) $w_D = 15psf$
$w_L = 40psf$



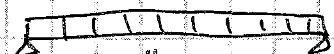
21' 3"
11 7/8 TJI 560 I-joists
(TJI 230 @ 19'10" span)
(TJI 110 @ 17'8" span)

(18) $w_D = 15psf\left(\frac{27'}{2}\right) = 202.5plf$
$w_L = 40psf\left(\frac{27'}{2}\right) = 540plf$



4x10
6'11" clear span max

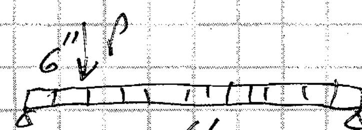
(19) $w_D = 15psf\left(\frac{19'}{2} + \frac{16'}{2}\right) = 262.5plf$
$w_L = 40psf\left(\frac{19'}{2} + \frac{16'}{2}\right) = 700plf$



4x10
6'1" max clear span

(20) $w_D = 15psf\left(\frac{39'}{2} + \frac{7'}{2} + 1'\right) + 12psf(9') = 468plf$
$w_L = 40plf$
$w_S = 25psf\left(\frac{39'}{2} + \frac{7'}{2}\right) = 575plf$

$P = 3160^{\#}DL + 4650^{\#}SL$ from girder



6'
1 3/4 x 11 7/8 LSL minimum
(3 1/2 for bearing condition)

FOR Lot 3 DATE 7-6-20
JOB BY MM

MYERS ENGINEERING

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

Wood Beam

Lic. # : KW-06008232 MYERS ENGINEERING

DESCRIPTION: 1. Header

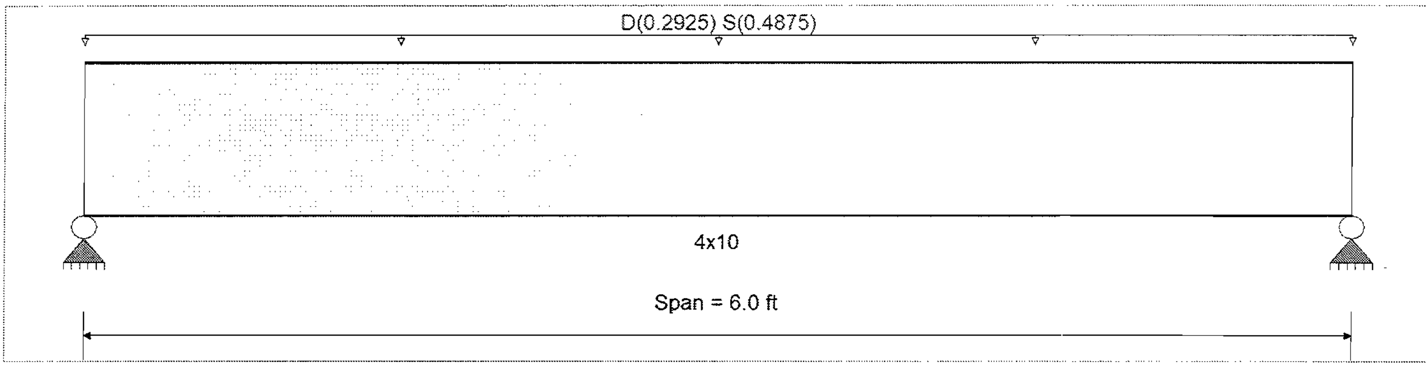
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	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	31.20 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 = 0.2925, S = 0.4875 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.679 : 1	Maximum Shear Stress Ratio	=	0.390 : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	843.89 psi		=	80.72 psi
	=	1,242.00 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	0.039 in	Ratio =	1859 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.062 in	Ratio =	1162 >=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	2.340	2.340
Overall MINimum	1.463	1.463
D Only	0.878	0.878
+D+L	0.878	0.878
+D+S	2.340	2.340
+D+0.750L	0.878	0.878
+D+0.750L+0.750S	1.974	1.974
+0.60D	0.527	0.527
S Only	1.463	1.463

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Wood Beam

DESCRIPTION: 2. Beam over 2 Car Bay

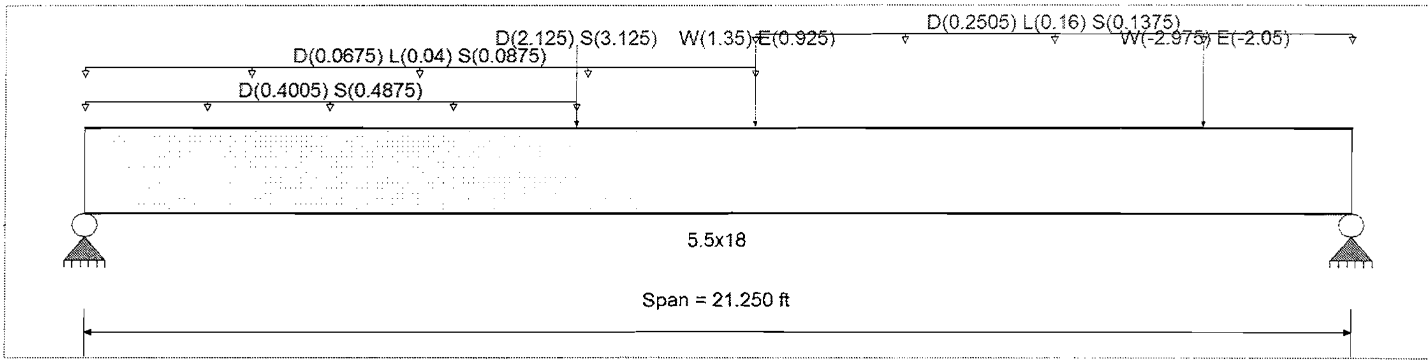
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	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 = 0.4005, S = 0.4875 k/ft, Extent = 0.0 -->> 8.250 ft, Tributary Width = 1.0 ft, (Wall & Long Span Roof)
- Uniform Load : D = 0.06750, L = 0.040, S = 0.08750 k/ft, Extent = 0.0 -->> 11.250 ft, Tributary Width = 1.0 ft, (Low Roof)
- Uniform Load : D = 0.2505, L = 0.160, S = 0.1375 k/ft, Extent = 11.250 -->> 21.250 ft, Tributary Width = 1.0 ft, (Cantilever Floor & Low Roof)
- Point Load : D = 2.125, S = 3.125 k @ 8.250 ft, (Roof Girder Truss)
- Point Load : W = 1.350, E = 0.9250 k @ 11.250 ft, (Overturning Load at Perforated Shearwall)
- Point Load : W = -2.975, E = -2.050 k @ 18.750 ft, (Overtrning at Corner Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.888 : 1	Maximum Shear Stress Ratio	=	0.486 : 1
Section used for this span		**5.5x18**	Section used for this span		**5.5x18**
	=	2,332.92 psi		=	148.09 psi
	=	2,628.57 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.221 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	0.470 in	Ratio =	542 >=360
Max Upward Transient Deflection	-0.024 in	Ratio =	10593 >=360
Max Downward Total Deflection	0.883 in	Ratio =	288 >=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	11.311	7.929
Overall MINimum	-0.285	1.319
D Only	5.111	3.583
+D+L	5.818	4.926
+D+S	11.311	6.889
+D+0.750L	5.641	4.590
+D+0.750L+0.750S	10.291	7.069
+D+0.60W	5.282	2.437
+D-0.60W	4.939	4.729

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DESCRIPTION: 2. Beam over 2 Car Bay

Vertical Reactions

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

Load Combination	Support 1	Support 2
+D+0.70E	5.246	2.660
+D-0.70E	4.975	4.506
+D+0.750L+0.450W	5.769	3.730
+D+0.750L-0.450W	5.513	5.450
+D+0.750L+0.750S+0.450W	10.420	6.210
+D+0.750L+0.750S-0.450W	10.163	7.929
+D+0.750L+0.750S+0.5250E	10.393	6.377
+D+0.750L+0.750S-0.5250E	10.189	7.762
+0.60D+0.60W	3.237	1.004
+0.60D-0.60W	2.895	3.296
+0.60D+0.70E	3.202	1.226
+0.60D-0.70E	2.930	3.073
L Only	0.707	1.343
S Only	6.200	3.306
W Only	0.285	-1.910
-W	-0.285	1.910
E Only	0.194	-1.319
E Only * -1.0	-0.194	1.319

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Gig Harbor, WA 98335

Wood Beam

File: 3404 72nd PL SE.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

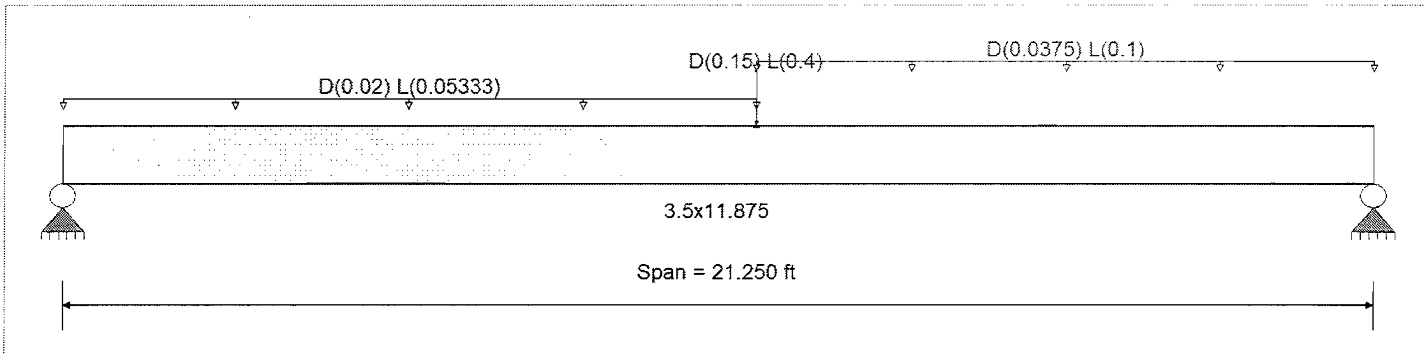
DESCRIPTION: 3. Floor beam at Bedrrom 2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2325 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	Fb -	2325 psi	Ebend- xx	1550 ksi
		Fc - Prll	2170 psi	Eminbend - xx	787.815 ksi
Wood Species	: Trus Joist	Fc - Perp	900 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310 psi		
		Ft	1070 psi	Density	45.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.020, L = 0.05333 k/ft, Extent = 0.0 -->> 11.250 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.03750, L = 0.10 k/ft, Extent = 11.250 -->> 21.250 ft, Tributary Width = 1.0 ft
Point Load : D = 0.150, L = 0.40 k @ 11.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.548: 1**	Maximum Shear Stress Ratio	=	**0.167 : 1**
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	1,274.05psi		=	51.72 psi
	=	2,325.00psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	11.245ft	Location of maximum on span	=	20.319 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.637 in	Ratio =	400 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.876 in	Ratio =	291 >=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	1.189	1.561
Overall MINimum	0.865	1.135
D Only	0.324	0.426
+D+L	1.189	1.561
+D+S	0.324	0.426
+D+0.750L	0.973	1.277
+D+0.750L+0.750S	0.973	1.277
+0.60D	0.195	0.255
L Only	0.865	1.135
S Only		

MYERS ENGINEERING

Mark Myers, PE
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Gig Harbor, WA 98335

Wood Beam

DESCRIPTION: 4. Beam over Garage at Grid 7

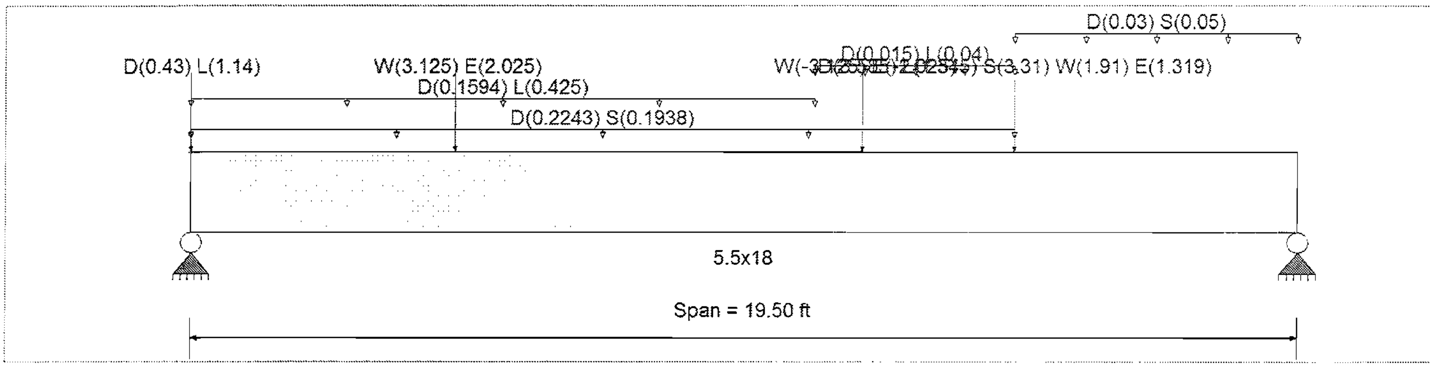
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	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 = 0.2243, S = 0.1938 k/ft, Extent = 0.0 -->> 14.50 ft, Tributary Width = 1.0 ft, (Wall & Upper Roof)
- Uniform Load : D = 0.1594, L = 0.4250 k/ft, Extent = 0.0 -->> 11.0 ft, Tributary Width = 1.0 ft, (Long Span Floor)
- Uniform Load : D = 0.0150, L = 0.040 k/ft, Extent = 11.0 -->> 14.50 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.430, L = 1.140 k @ 0.0 ft, (Beam 3)
- Point Load : D = 3.585, L = 1.345, S = 3.310, W = 1.910, E = 1.319 k @ 14.50 ft, (Beam 2)
- Uniform Load : D = 0.030, S = 0.050 k/ft, Extent = 14.50 -->> 19.50 ft, Tributary Width = 1.0 ft, (Lower Roof)
- Point Load : W = 3.125, E = 2.025 k @ 4.670 ft, (Overturning Load)
- Point Load : W = -3.125, E = -2.025 k @ 11.833 ft, (Overturning Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.737: 1**	Maximum Shear Stress Ratio	=	**0.449 : 1**
Section used for this span		**5.5x18**	Section used for this span		**5.5x18**
	=	1,954.01 psi		=	136.89 psi
	=	2,651.26 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	10.675ft	Location of maximum on span	=	18.005 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.247 in	Ratio =	945 >=360
Max Upward Transient Deflection	-0.037 in	Ratio =	6310 >=360
Max Downward Total Deflection	0.713 in	Ratio =	328 >=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	11.077	9.260
Overall MINimum	-1.638	-0.272
D Only	4.689	4.535
+D+L	9.578	6.945
+D+S	7.335	8.259
+D+0.750L	8.356	6.342
+D+0.750L+0.750S	10.341	9.135

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DESCRIPTION: 4. Beam over Garage at Grid 7

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.60W	5.671	4.698
+D-0.60W	3.706	4.371
+D+0.70E	5.446	4.700
+D-0.70E	3.931	4.369
+D+0.750L+0.450W	9.093	6.465
+D+0.750L-0.450W	7.619	6.220
+D+0.750L+0.750S+0.450W	11.077	9.258
+D+0.750L+0.750S-0.450W	9.604	9.013
+D+0.750L+0.750S+0.5250E	10.909	9.260
+D+0.750L+0.750S-0.5250E	9.772	9.011
+0.60D+0.60W	3.796	2.884
+0.60D-0.60W	1.831	2.557
+0.60D+0.70E	3.571	2.887
+0.60D-0.70E	2.056	2.555
L Only	4.890	2.410
S Only	2.646	3.724
W Only	1.638	0.272
-W	-1.638	-0.272
E Only	1.082	0.237
E Only * -1.0	-1.082	-0.237

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

Wood Beam

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DESCRIPTION: 5. Beams over Dining Room

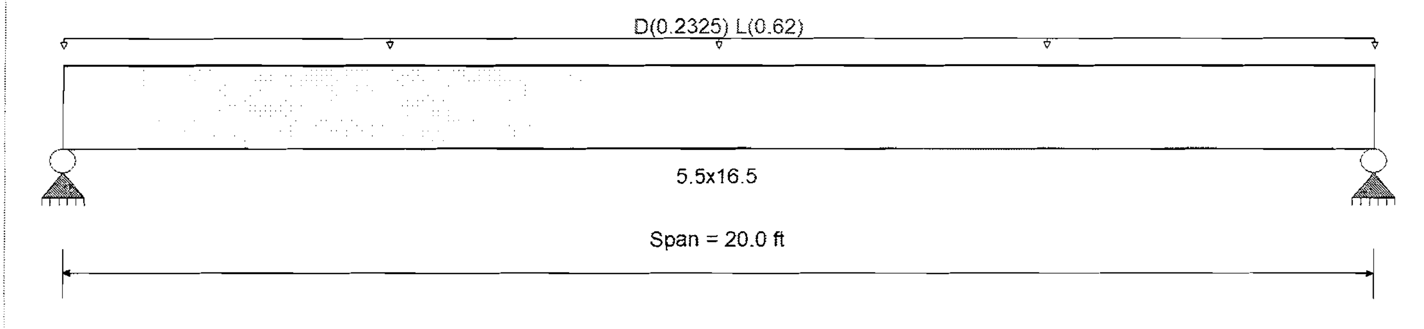
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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 = 0.2325, L = 0.620 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.884: 1	Maximum Shear Stress Ratio	=	0.462 : 1
Section used for this span		**5.5x16.5**	Section used for this span		**5.5x16.5**
	=	2,049.59 psi		=	122.40 psi
	=	2,319.71 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	10.000 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	0.606 in	Ratio =	396 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.833 in	Ratio =	288 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.525	8.525
Overall MINimum	6.200	6.200
D Only	2.325	2.325
+D+L	8.525	8.525
+D+S	2.325	2.325
+D+0.750L	6.975	6.975
+D+0.750L+0.750S	6.975	6.975
+0.60D	1.395	1.395
L Only	6.200	6.200
S Only		

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Wood Beam

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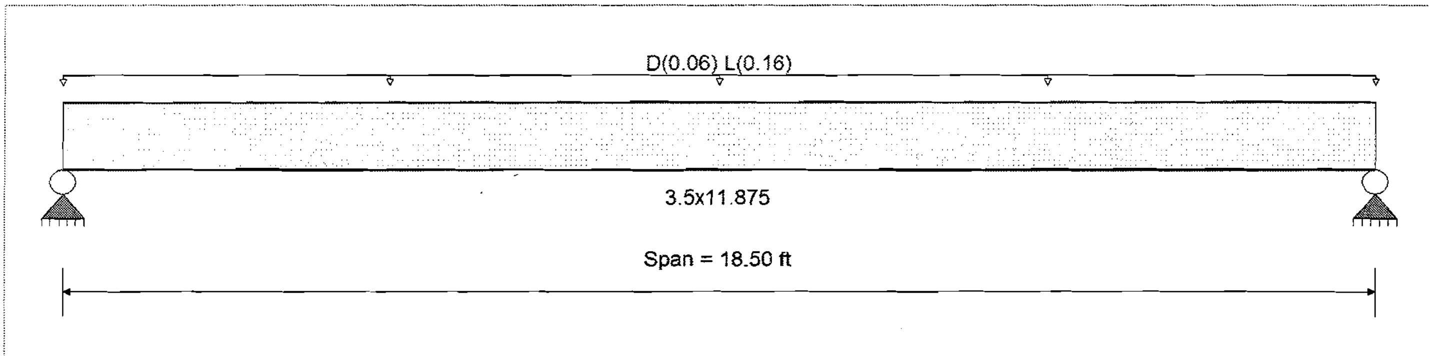
DESCRIPTION: 6. Rim Beam at Stair

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2325 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2325 psi	Ebend- xx	1550 ksi
		Fc - Prll	2170 psi	Eminbend - xx	787.815 ksi
Wood Species	: Trus Joist	Fc - Perp	900 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310 psi		
		Ft	1070 psi	Density	45.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.060, L = 0.160 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.591**: 1	Maximum Shear Stress Ratio	=	**0.213** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	1,373.01 psi		=	65.94 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.250 ft	Location of maximum on span	=	17.555 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.560 in	Ratio =	396 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.770 in	Ratio =	288 >=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	2.035	2.035
Overall MINimum	1.480	1.480
D Only	0.555	0.555
+D+L	2.035	2.035
+D+S	0.555	0.555
+D+0.750L	1.665	1.665
+D+0.750L+0.750S	1.665	1.665
+0.60D	0.333	0.333
L Only	1.480	1.480
S Only		

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DESCRIPTION: 7. Beam at Grid 2 over Kitchen

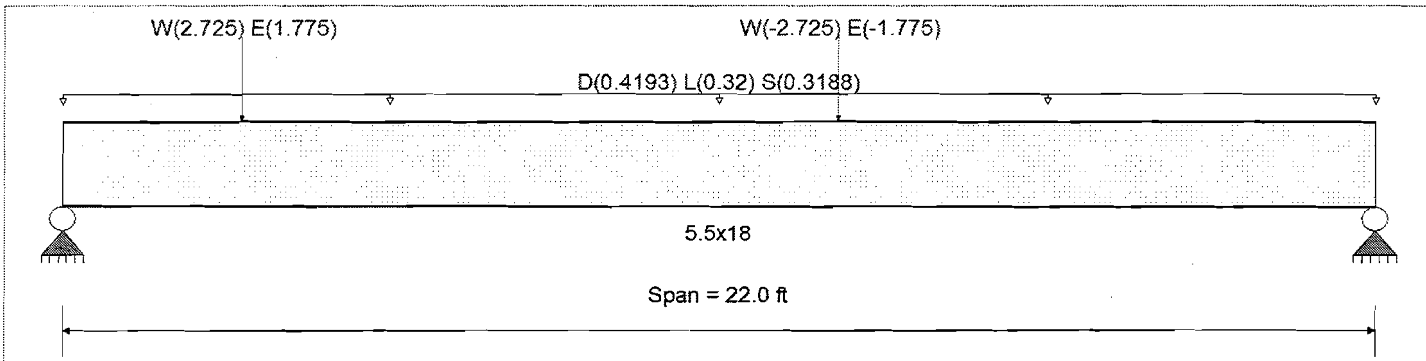
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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 = 0.4193, L = 0.320, S = 0.3188 , Tributary Width = 1.0 ft
Point Load : W = 2.725, E = 1.775 k @ 3.0 ft, (Overturning load from wall above)
Point Load : W = -2.725, E = -1.775 k @ 13.0 ft, (Overturning load from wall above)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.838** 1	Maximum Shear Stress Ratio	=	**0.427** : 1
Section used for this span		**5.5x18**	Section used for this span		**5.5x18**
	=	2,196.09 psi		=	130.06 psi
	=	2,619.47 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	11.000 ft	Location of maximum on span	=	20.555 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.353 in	Ratio =	748 >=360
Max Upward Transient Deflection	-0.125 in	Ratio =	2113 >=360
Max Downward Total Deflection	1.044 in	Ratio =	252 >=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	10.440	10.440
Overall MINimum	-1.239	0.807
D Only	4.612	4.612
+D+L	8.132	8.132
+D+S	8.119	8.119
+D+0.750L	7.252	7.252
+D+0.750L+0.750S	9.882	9.882
+D+0.60W	5.355	3.869
+D-0.60W	3.869	5.355
+D+0.70E	5.177	4.048
+D-0.70E	4.048	5.177
+D+0.750L+0.450W	7.810	6.695
+D+0.750L-0.450W	6.695	7.810

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DESCRIPTION: 7. Beam at Grid 2 over Kitchen

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W	10.440	9.325
+D+0.750L+0.750S-0.450W	9.325	10.440
+D+0.750L+0.750S+0.5250E	10.306	9.459
+D+0.750L+0.750S-0.5250E	9.459	10.306
+0.60D+0.60W	3.511	2.024
+0.60D-0.60W	2.024	3.511
+0.60D+0.70E	3.332	2.203
+0.60D-0.70E	2.203	3.332
L Only	3.520	3.520
S Only	3.507	3.507
W Only	1.239	-1.239
-W	-1.239	1.239
E Only	0.807	-0.807
E Only * -1.0	-0.807	0.807

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Wood Beam

DESCRIPTION: 8. Beam at Grid 2 over Guest

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

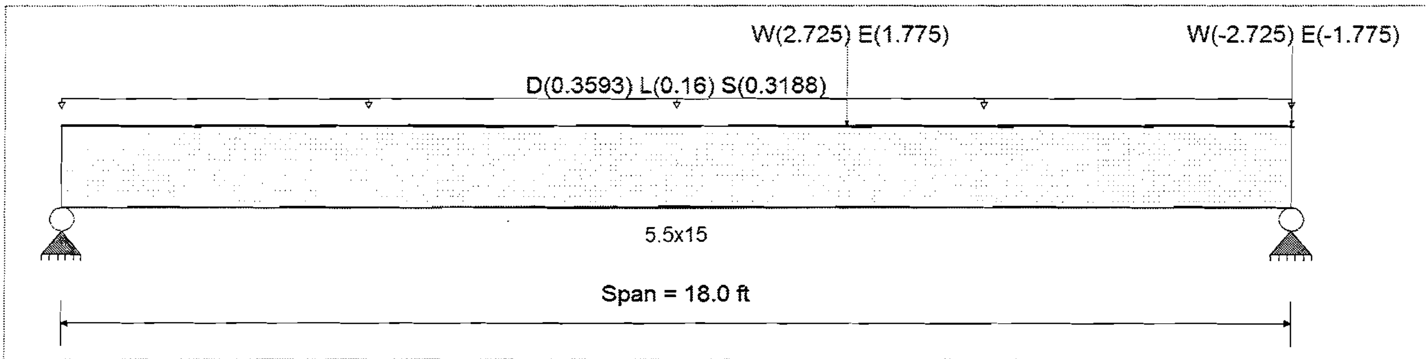
Wood Species : DF/DF
Wood Grade : 24F - V4

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

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

E : Modulus of Elasticity

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



Applied Loads

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

Uniform Load : D = 0.3593, L = 0.160, S = 0.3188 , Tributary Width = 1.0 ft
Point Load : W = 2.725, E = 1.775 k @ 11.50 ft, (Overturning load from wall above)
Point Load : W = -2.725, E = -1.775 k @ 18.0 ft, (Overturning load from wall above)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.622** 1	Maximum Shear Stress Ratio	=	**0.332** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	1,692.81 psi		=	101.25 psi
	=	2,721.74 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.752 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.272 in	Ratio =	794 >=360
Max Upward Transient Deflection	-0.186 in	Ratio =	1159 >=360
Max Downward Total Deflection	0.696 in	Ratio =	310 >=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	6.908	6.908
Overall MINimum	-0.984	0.641
D Only	3.234	3.234
+D+L	4.674	4.674
+D+S	6.103	6.103
+D+0.750L	4.314	4.314
+D+0.750L+0.750S	6.466	6.466
+D+0.60W	3.824	2.643
+D-0.60W	2.643	3.824
+D+0.70E	3.682	2.785
+D-0.70E	2.785	3.682
+D+0.750L+0.450W	4.757	3.871
+D+0.750L-0.450W	3.871	4.757

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DESCRIPTION: 8. Beam at Grid 2 over Guest

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W	6.908	6.023
+D+0.750L+0.750S-0.450W	6.023	6.908
+D+0.750L+0.750S+0.5250E	6.802	6.129
+D+0.750L+0.750S-0.5250E	6.129	6.802
+0.60D+0.60W	2.531	1.350
+0.60D-0.60W	1.350	2.531
+0.60D+0.70E	2.389	1.492
+0.60D-0.70E	1.492	2.389
L Only	1.440	1.440
S Only	2.869	2.869
W Only	0.984	-0.984
-W	-0.984	0.984
E Only	0.641	-0.641
E Only * -1.0	-0.641	0.641

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Wood Beam

DESCRIPTION: 9. Kitchen Window Header

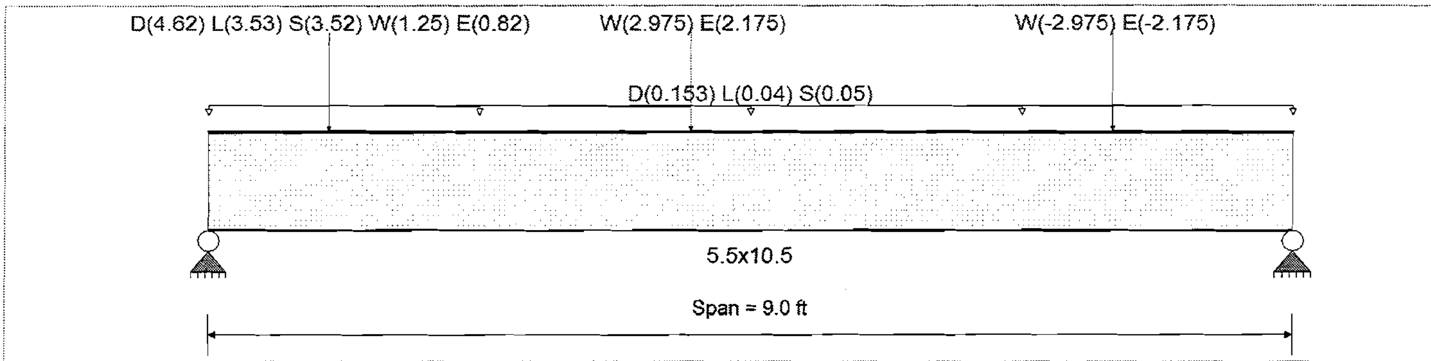
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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 = 0.1530, L = 0.040, S = 0.050 , Tributary Width = 1.0 ft
Point Load : D = 4.620, L = 3.530, S = 3.520, W = 1.250, E = 0.820 k @ 1.0 ft
Point Load : W = 2.975, E = 2.175 k @ 4.0 ft
Point Load : W = -2.975, E = -2.175 k @ 7.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.439 : 1**	Maximum Shear Stress Ratio	=	**0.819 : 1**
Section used for this span		**5.5x10.5**	Section used for this span		**5.5x10.5**
	=	1,683.90 psi		=	249.63 psi
	=	3,840.00 psi		=	304.75 psi
Load Combination		+1.105D+0.750L+0.750S+1.575E	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	3.974 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	0.054 in	Ratio =	1995 >=360
Max Upward Transient Deflection	-0.054 in	Ratio =	1995 >=360
Max Downward Total Deflection	0.150 in	Ratio =	719 >=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	10.820	2.551
Overall MINimum	-2.268	0.755
D Only	4.795	1.202
+D+L	8.113	1.774
+D+S	8.149	1.818
+D+0.750L	7.284	1.631
+D+0.750L+0.750S	9.799	2.093
+D+0.60W	6.156	0.591
+D-0.60W	3.434	1.813
+D+0.70E	5.897	0.674
+D-0.70E	3.693	1.730
+D+0.750L+0.450W	8.304	1.173

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DESCRIPTION: 9. Kitchen Window Header

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L-0.450W	6.263	2.089
+D+0.750L+0.750S+0.450W	10.820	1.635
+D+0.750L+0.750S-0.450W	8.778	2.551
+D+0.750L+0.750S+0.5250E	10.626	1.697
+D+0.750L+0.750S-0.5250E	8.972	2.489
+0.60D+0.60W	4.238	0.110
+0.60D-0.60W	1.516	1.332
+0.60D+0.70E	3.979	0.193
+0.60D-0.70E	1.775	1.249
L Only	3.318	0.572
S Only	3.354	0.616
W Only	2.268	-1.018
-W	-2.268	1.018
E Only	1.575	-0.755
E Only * -1.0	-1.575	0.755

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DESCRIPTION: 10. Guest Rm Window Header

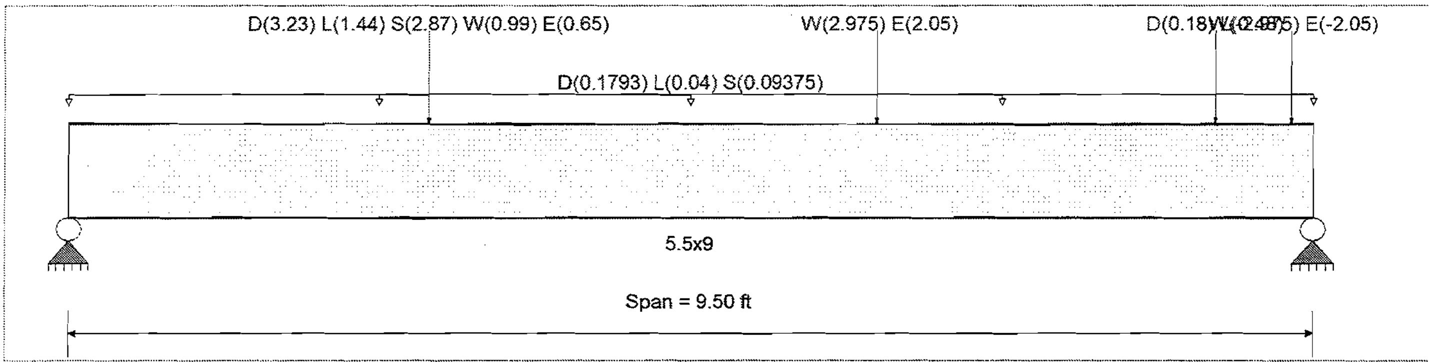
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1793, L = 0.040, S = 0.09375 , Tributary Width = 1.0 ft
Point Load : D = 3.230, L = 1.440, S = 2.870, W = 0.990, E = 0.650 k @ 2.750 ft, (Load from Beam 8)
Point Load : D = 0.180, L = 0.480 k @ 8.750 ft, (Shower Beam)
Point Load : W = 2.975, E = 2.050 k @ 6.170 ft, (Overturning from wall above)
Point Load : W = -2.975, E = -2.050 k @ 9.330 ft, (Overturning from wall above)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.896 1	Maximum Shear Stress Ratio	=	**0.573** : 1
Section used for this span		**5.5x9**	Section used for this span		**5.5x9**
	=	2,474.28 psi		=	174.51 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	=	2.774 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	0.166 in	Ratio =	684 >=360
Max Upward Transient Deflection	-0.166 in	Ratio =	684 >=360
Max Downward Total Deflection	0.426 in	Ratio =	267 >=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	6.724	4.013
Overall MINimum	-1.693	0.494
D Only	3.161	1.952
+D+L	4.412	3.001
+D+S	5.645	3.229
+D+0.750L	4.099	2.739
+D+0.750L+0.750S	5.963	3.696
+D+0.60W	4.177	1.531
+D-0.60W	2.145	2.374
+D+0.70E	3.962	1.607
+D-0.70E	2.360	2.298

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DESCRIPTION: 10. Guest Rm Window Header

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.450W	4.861	2.423
+D+0.750L-0.450W	3.337	3.056
+D+0.750L+0.750S+0.450W	6.724	3.380
+D+0.750L+0.750S-0.450W	5.201	4.013
+D+0.750L+0.750S+0.5250E	6.563	3.437
+D+0.750L+0.750S-0.5250E	5.362	3.955
+0.60D+0.60W	2.912	0.750
+0.60D-0.60W	0.881	1.593
+0.60D+0.70E	2.697	0.826
+0.60D-0.70E	1.096	1.517
L Only	1.251	1.049
S Only	2.485	1.276
W Only	1.693	-0.703
-W	-1.693	0.703
E Only	1.144	-0.494
E Only * -1.0	-1.144	0.494

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Wood Beam

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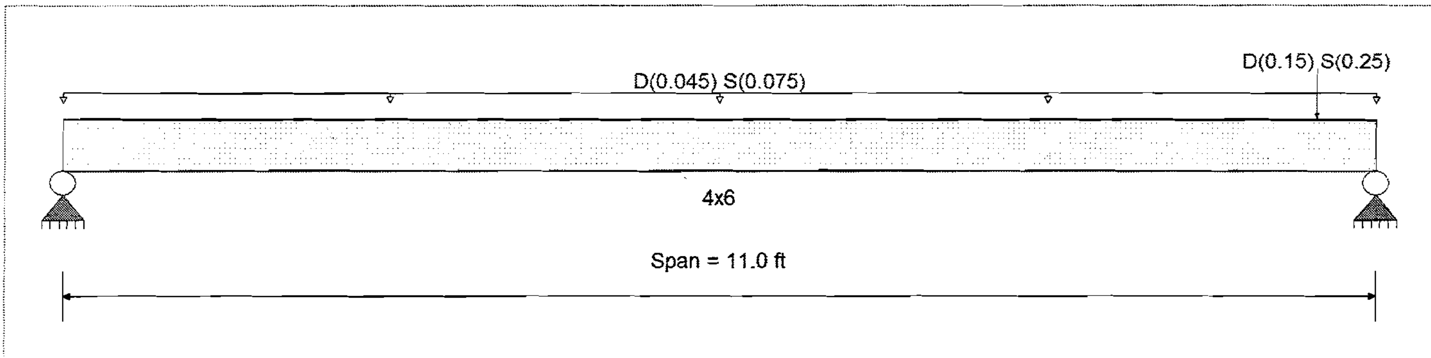
DESCRIPTION: 11. Front Porch Roof Beam

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900 psi	Ebend- xx	1600 ksi
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	180 psi		
		Ft	575 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
Point Load : D = 0.150, S = 0.250 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.969 : 1**	Maximum Shear Stress Ratio	=	**0.372 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
	=	1,303.22 psi		=	77.05 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.661 ft	Location of maximum on span	=	10.558 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.341 in	Ratio =	386 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.546 in	Ratio =	241 >=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	0.678	1.042
Overall MINimum	0.424	0.651
D Only	0.254	0.391
+D+L	0.254	0.391
+D+S	0.678	1.042
+D+0.750L	0.254	0.391
+D+0.750L+0.750S	0.572	0.879
+0.60D	0.153	0.234
S Only	0.424	0.651

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Wood Beam

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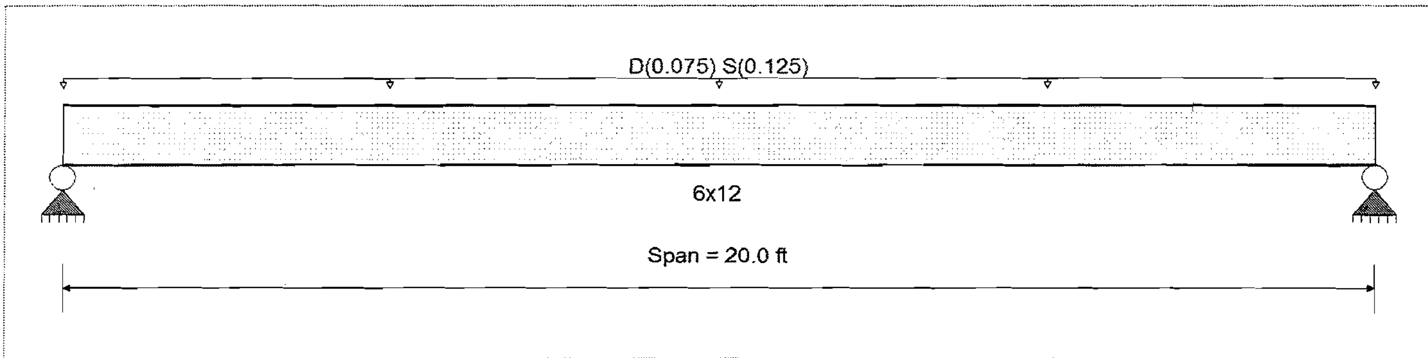
DESCRIPTION: 12. Back Porch Roof Beam

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1350 psi	Ebend- xx	1600 ksi
		Fc - Prll	925 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.1	Fv	170 psi		
		Ft	675 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 = 0.0750, S = 0.1250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.638** 1	Maximum Shear Stress Ratio	=	**0.220** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	989.86 psi		=	42.93 psi
	=	1,552.50 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.000 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	0.406 in	Ratio =	591 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.649 in	Ratio =	369 >=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	2.000	2.000
Overall MINimum	1.250	1.250
D Only	0.750	0.750
+D+L	0.750	0.750
+D+S	2.000	2.000
+D+0.750L	0.750	0.750
+D+0.750L+0.750S	1.688	1.688
+0.60D	0.450	0.450
S Only	1.250	1.250

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Wood Beam

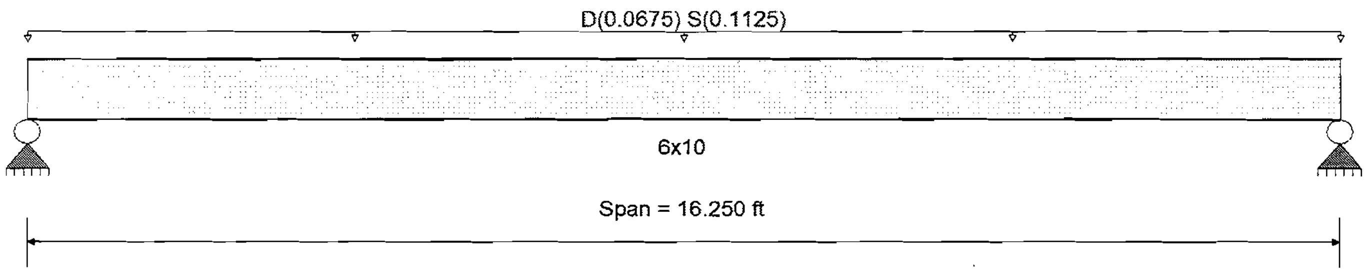
DESCRIPTION: 13. 2 Car Door Header

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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 = 0.06750, S = 0.1125 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.856**: 1	Maximum Shear Stress Ratio	=	**0.194** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	861.81 psi		=	38.00 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125ft	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	0.348 in	Ratio =	561 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.556 in	Ratio =	350 >=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	1.463	1.463
Overall MINimum	0.914	0.914
D Only	0.548	0.548
+D+L	0.548	0.548
+D+S	1.463	1.463
+D+0.750L	0.548	0.548
+D+0.750L+0.750S	1.234	1.234
+0.60D	0.329	0.329
S Only	0.914	0.914

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DESCRIPTION: 14. 3rd Car Door Header

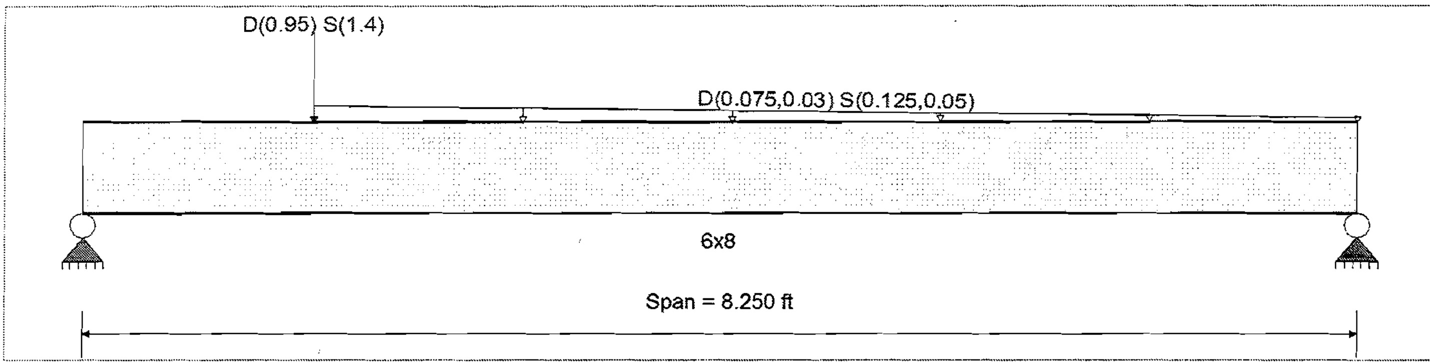
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	875.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	600.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	170.0 psi		
		Ft	425.0 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

Varying Uniform Load : D= 0.0750->0.030, S= 0.1250->0.050 k/ft, Extent = 1.50 -->> 8.250 ft, Trib Width = 1.0 ft

Point Load : D = 0.950, S = 1.40 k @ 1.50 ft, (Hip Girder)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.820** : 1	Maximum Shear Stress Ratio	=	**0.440** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
	=	825.56 psi		=	85.98 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	1.566 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	0.096 in	Ratio =	1034 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.158 in	Ratio =	627 >=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	2.365	0.930
Overall MINimum	1.422	0.569
D Only	0.943	0.361
+D+L	0.943	0.361
+D+S	2.365	0.930
+D+0.750L	0.943	0.361
+D+0.750L+0.750S	2.009	0.788
+0.60D	0.566	0.217
S Only	1.422	0.569

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Wood Beam

File: 3404 72nd PL SE.ec6

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DESCRIPTION: 15. Gable beam at Back Porch

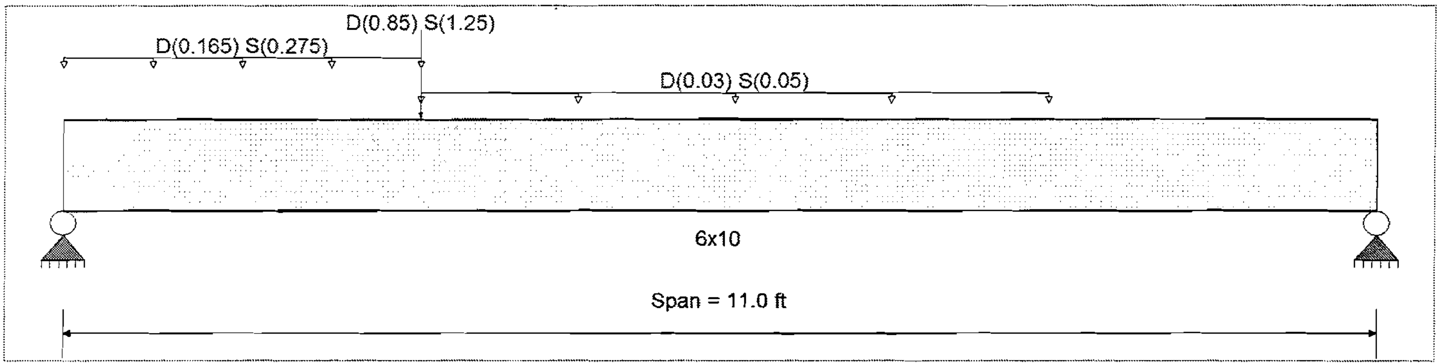
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	875.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	600.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	170.0 psi		
		Ft	425.0 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 = 3.0 -->> 8.250 ft, Tributary Width = 1.0 ft
Point Load : D = 0.850, S = 1.250 k @ 3.0 ft, (Hip Girder)
Uniform Load : D = 0.1650, S = 0.2750 k/ft, Extent = 0.0 -->> 3.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.956 : 1**	Maximum Shear Stress Ratio	=	**0.373 : 1**
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	961.91 psi		=	72.83 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.011 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	0.141 in	Ratio =	935 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.233 in	Ratio =	566 >=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	2.873	0.968
Overall MINimum	1.750	0.588
D Only	1.123	0.380
+D+L	1.123	0.380
+D+S	2.873	0.968
+D+0.750L	1.123	0.380
+D+0.750L+0.750S	2.435	0.821
+0.60D	0.674	0.228
S Only	1.750	0.588

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Wood Beam

DESCRIPTION: 16. Dining Rm window header

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

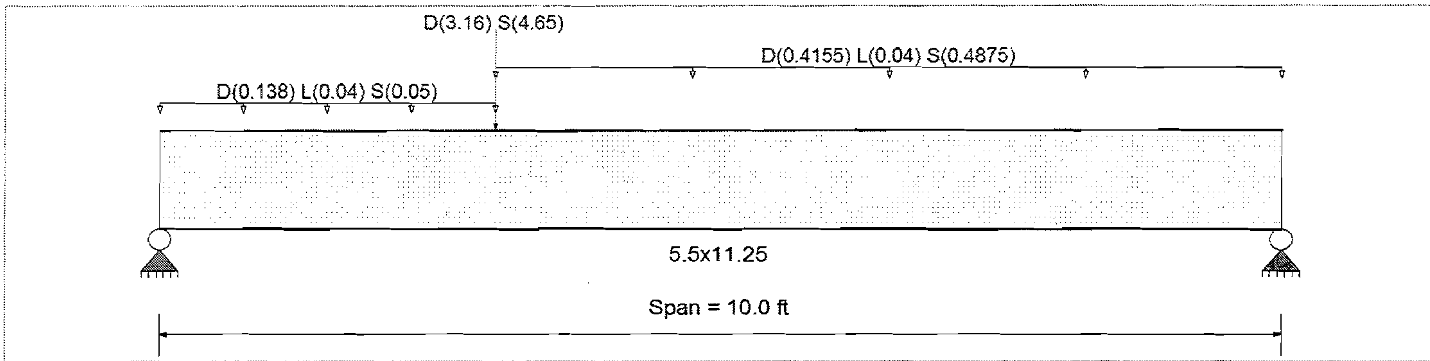
Wood Species : DF/DF
Wood Grade : 24F - V4

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

Fb +	2400	psi
Fb -	1850	psi
Fc - Prll	1650	psi
Fc - Perp	650	psi
Fv	265	psi
Ft	1100	psi

E : Modulus of Elasticity

Ebend- xx	1800	ksi
Eminbend - xx	950	ksi
Ebend- yy	1600	ksi
Eminbend - yy	850	ksi
Density	31.21	pcf



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.1380, L = 0.040, S = 0.050 k/ft, Extent = 0.0 -->> 3.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.4155, L = 0.040, S = 0.4875 k/ft, Extent = 3.0 -->> 10.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 3.160, S = 4.650 k @ 3.0 ft, (Girder at Upper Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.885 1	Maximum Shear Stress Ratio	=	0.635 : 1
Section used for this span		**5.5x11.25**	Section used for this span		**5.5x11.25**
	=	2,443.49psi		=	193.63 psi
	=	2,760.00psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.029ft	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	0.191 in	Ratio =	628 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.338 in	Ratio =	355 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.159	6.536
Overall MINimum	4.577	3.636
D Only	3.582	2.901
+D+L	3.782	3.101
+D+S	8.159	6.536
+D+0.750L	3.732	3.051
+D+0.750L+0.750S	7.165	5.777
+0.60D	2.149	1.740
L Only	0.200	0.200
S Only	4.577	3.636

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Steel Beam

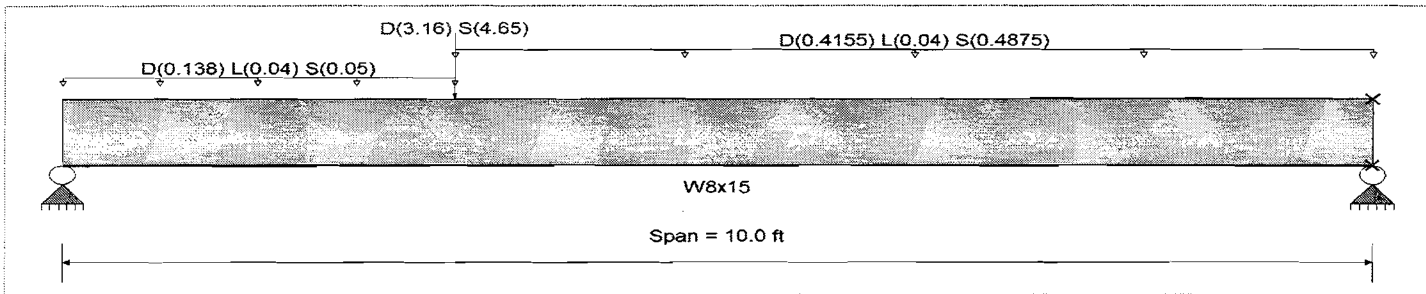
DESCRIPTION: 16. Dining Rm window header

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.1380, L = 0.040, S = 0.050 k/ft, Extent = 0.0 -->> 3.0 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.4155, L = 0.040, S = 0.4875 k/ft, Extent = 3.0 -->> 10.0 ft, Tributary Width = 1.0 ft

Point Load : D = 3.160, S = 4.650 k @ 3.0 ft, (Girder at Upper Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.696** : 1	Maximum Shear Stress Ratio =	**0.205** : 1
Section used for this span	**W8x15**	Section used for this span	**W8x15**
Ma : Applied	23.630 k-ft	Va : Applied	8.159 k
Mn / Omega : Allowable	33.932 k-ft	Vn/Omega : Allowable	39.739 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	3.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection			
Max Downward Transient Deflection	0.161 in	Ratio =	745 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.285 in	Ratio =	422 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.2846	4.771		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.159	6.536
Overall MINimum	0.200	0.200
D Only	3.582	2.901
+D+L	3.782	3.101
+D+S	8.159	6.536
+D+0.750L	3.732	3.051
+D+0.750L+0.750S	7.165	5.777
+0.60D	2.149	1.740
L Only	0.200	0.200
S Only	4.577	3.636

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Steel Beam

DESCRIPTION: 16. Dining Rm window header

Steel Section Properties : W8x15

Depth	=	8.110 in	I xx	=	48.00 in^4	J	=	0.137 in^4
Web Thick	=	0.245 in	S xx		11.80 in^3	Cw	=	51.80 in^6
Flange Width	=	4.015 in	R xx	=	3.290 in			
Flange Thick	=	0.315 in	Zx	=	13.600 in^3			
Area	=	4.440 in^2	I yy	=	3.410 in^4			
Weight	=	15.000 plf	S yy	=	1.700 in^3	Wno	=	7.810 in^2
Kdesign	=	0.615 in	R yy	=	0.876 in	Sw	=	2.470 in^4
K1	=	0.563 in	Zy	=	2.670 in^3	Qf	=	2.310 in^3
rts	=	1.060 in				Qw	=	6.640 in^3
Ycg	=	4.055 in						

17

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"[1]	20'-2"	17'-8"	16'-1"[1]	14'-4"[1]
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"[1]
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"[1]
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"[1]
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"[1]
14"	110	22'-10"	20'-11"	19'-2"	17'-2"[1]	22'-2"	19'-2"	17'-6"[1]	15'-0"[1]
	210	23'-11"	21'-10"	20'-8"	18'-10"[1]	23'-11"	21'-1"	19'-2"[1]	16'-7"[1]
	230	24'-8"	22'-6"	21'-2"	19'-9"[1]	24'-8"	22'-2"	20'-3"[1]	17'-6"[1]
	360	26'-0"	23'-8"	22'-4"	20'-9"[1]	26'-0"	23'-8"	22'-4"[1]	17'-10"[1]
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"[1]	20'-11"[1]
16"	110	25'-4"	22'-6"	20'-7"[1]	18'-1"[1]	23'-9"	20'-7"[1]	18'-9"[1]	15'-0"[1]
	210	26'-6"	24'-3"	22'-6"[1]	19'-11"[1]	26'-0"	22'-6"[1]	20'-7"[1]	16'-7"[1]
	230	27'-3"	24'-10"	23'-6"	21'-1"[1]	***27'-3"***	23'-9"	21'-8"[1]	17'-6"[1]
	360	28'-9"	26'-3"	24'-8"[1]	21'-5"[1]	***28'-9"***	26'-3"[1]	22'-4"[1]	17'-10"[1]
	560	32'-8"	29'-8"	28'-0"	25'-2"[1]	***32'-8"***	***29'-8"***	26'-3"[1]	20'-11"[1]

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"[1]	20'-5"	17'-8"	16'-1"[1]	14'-4"[1]
	210	23'-4"	21'-2"	19'-4"	17'-3"[1]	***22'-4"***	19'-4"	17'-8"	15'-9"[1]
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"[1]
	360	25'-4"	23'-2"	21'-10"	20'-4"[1]	***25'-4"***	***23'-2"***	***21'-10"[1]***	17'-10"[1]
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"[1]
14"	110	24'-4"	21'-0"	19'-2"	17'-2"[1]	22'-2"	19'-2"	17'-6"[1]	15'-0"[1]
	210	26'-6"	23'-1"	21'-1"	18'-10"[1]	24'-4"	21'-1"	19'-2"[1]	16'-7"[1]
	230	27'-3"	24'-4"	22'-2"	19'-10"[1]	***25'-8"***	22'-2"	20'-3"[1]	17'-6"[1]
	360	28'-9"	26'-3"	24'-9"[1]	21'-5"[1]	***28'-9"***	***26'-3"[1]***	22'-4"[1]	17'-10"[1]
	560	32'-8"	29'-9"	28'-0"	25'-2"[1]	***32'-8"***	***29'-9"***	***26'-3"[1]***	20'-11"[1]
16"	110	26'-0"	22'-6"	20'-7"[1]	18'-1"[1]	23'-9"	20'-7"[1]	18'-9"[1]	15'-0"[1]
	210	28'-6"	24'-8"	22'-6"[1]	19'-11"[1]	26'-0"	22'-6"[1]	20'-7"[1]	16'-7"[1]
	230	30'-1"	26'-0"	23'-9"	21'-1"[1]	***27'-5"***	23'-9"	21'-8"[1]	17'-6"[1]
	360	31'-10"	29'-0"	26'-10"[1]	21'-5"[1]	***31'-10"***	***26'-10"[1]***	22'-4"[1]	17'-10"[1]
	560	36'-1"	32'-11"	31'-0"[1]	25'-2"[1]	***36'-1"***	***31'-6"[1]***	26'-3"[1]	20'-11"[1]

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is *less* than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

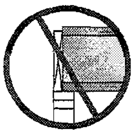
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

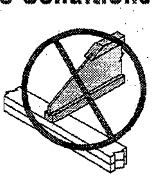
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

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

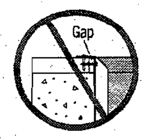
These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 18. Crawl Beam NOT at brg wall

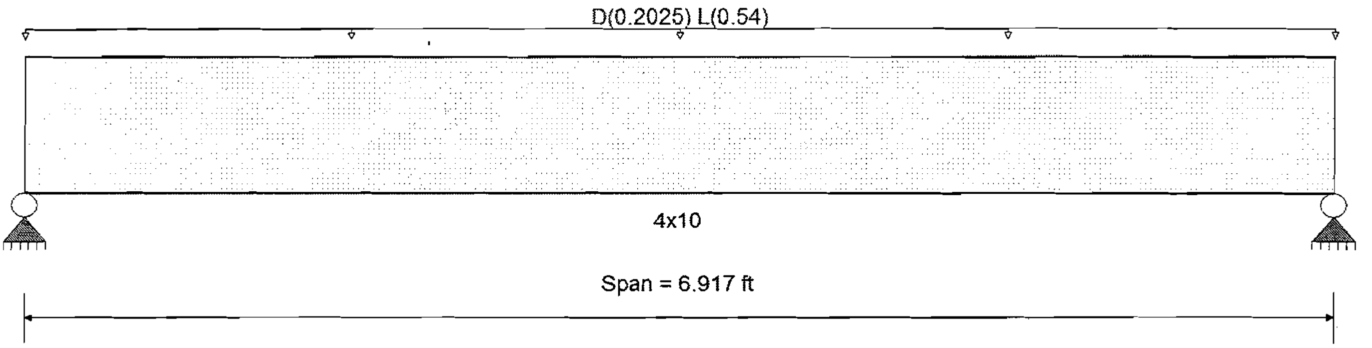
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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	31.20 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, L = 0.540 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.989 : 1**	Maximum Shear Stress Ratio	=	**0.516 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,067.64 psi		=	92.92 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.459 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	0.076 in	Ratio =	1095 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.104 in	Ratio =	796 >=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	2.568	2.568
Overall MINimum	1.868	1.868
D Only	0.700	0.700
+D+L	2.568	2.568
+D+S	0.700	0.700
+D+0.750L	2.101	2.101
+D+0.750L+0.750S	2.101	2.101
+0.60D	0.420	0.420
L Only	1.868	1.868
S Only		

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Wood Beam

Lic. # : KW-06008232 MYERS ENGINEERING

DESCRIPTION: 19. Crawl Beam at brg wall

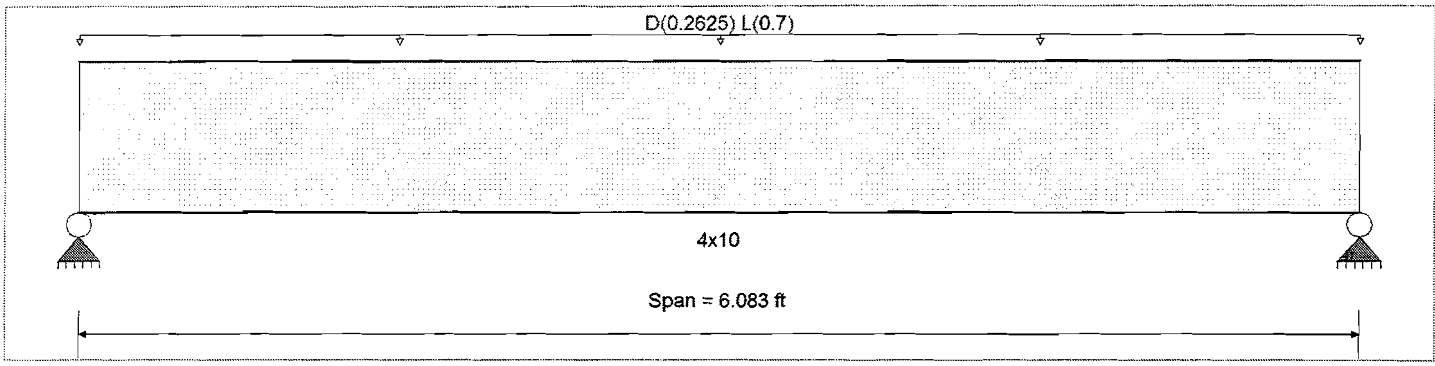
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
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	31.20 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 = 0.2625, L = 0.70 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.991**: 1	Maximum Shear Stress Ratio	=	**0.567** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,070.35psi		=	101.97 psi
	=	1,080.00psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.042ft	Location of maximum on span	=	5.328 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.059 in	Ratio =	1242 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.081 in	Ratio =	903 >=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	2.927	2.927
Overall MINimum	2.129	2.129
D Only	0.798	0.798
+D+L	2.927	2.927
+D+S	0.798	0.798
+D+0.750L	2.395	2.395
+D+0.750L+0.750S	2.395	2.395
+0.60D	0.479	0.479
L Only	2.129	2.129
S Only		

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Wood Beam

File: 3404 72nd PL SE.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

DESCRIPTION: 20. Rim Joist over Mud Rm

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

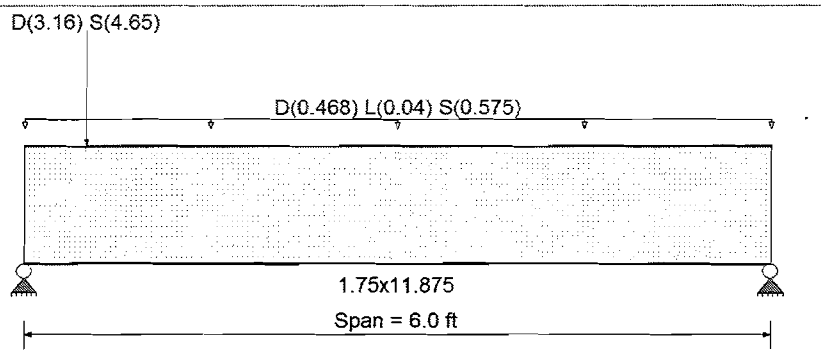
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

Wood Species : Trus Joist
Wood Grade : TimberStrand LSL 1.55E

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

Fb +	2325 psi	*E : Modulus of Elasticity*	
Fb -	2325 psi	Ebend- xx	1550 ksi
Fc - Prll	2170 psi	Eminbend - xx	787.815 ksi
Fc - Perp	900 psi		
Fv	310 psi		
Ft	1070 psi	Density	45.01 pcf



Applied Loads

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

Uniform Load : D = 0.4680, L = 0.040, S = 0.5750 , Tributary Width = 1.0 ft
Point Load : D = 3.160, S = 4.650 k @ 0.50 ft, (Girder at Upper Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.747: 1**	Maximum Shear Stress Ratio	=	**0.557 : 1**
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,998.27psi		=	198.64 psi
	=	2,673.75psi		=	356.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.387ft	Location of maximum on span	=	5.015ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.069 in	Ratio =	1049 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.121 in	Ratio =	593 >=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	10.288	3.780
Overall MINimum	5.988	2.113
D Only	4.301	1.667
+D+L	4.421	1.787
+D+S	10.288	3.780
+D+0.750L	4.391	1.757
+D+0.750L+0.750S	8.881	3.342
+0.60D	2.580	1.000
L Only	0.120	0.120
S Only	5.988	2.113

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)

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)

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)