MYERS ENGINEERING

LATERAL ANALYSIS & GRAVITY CALCULATIONS

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Date: 2020.08.03 14:49:34 -07'00'

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Project: Lot 4 - Walia
3406 72nd Place Southeast
Mercer Island, WA

August 3, 2020

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

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Gig Harbor, WA 98335
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Email: myengineer@centurytel.net

DESIGN LOADS:

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

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

WOODS :

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

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

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

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

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

TRUSSES:

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

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

ENGINEERED I-JOISTS

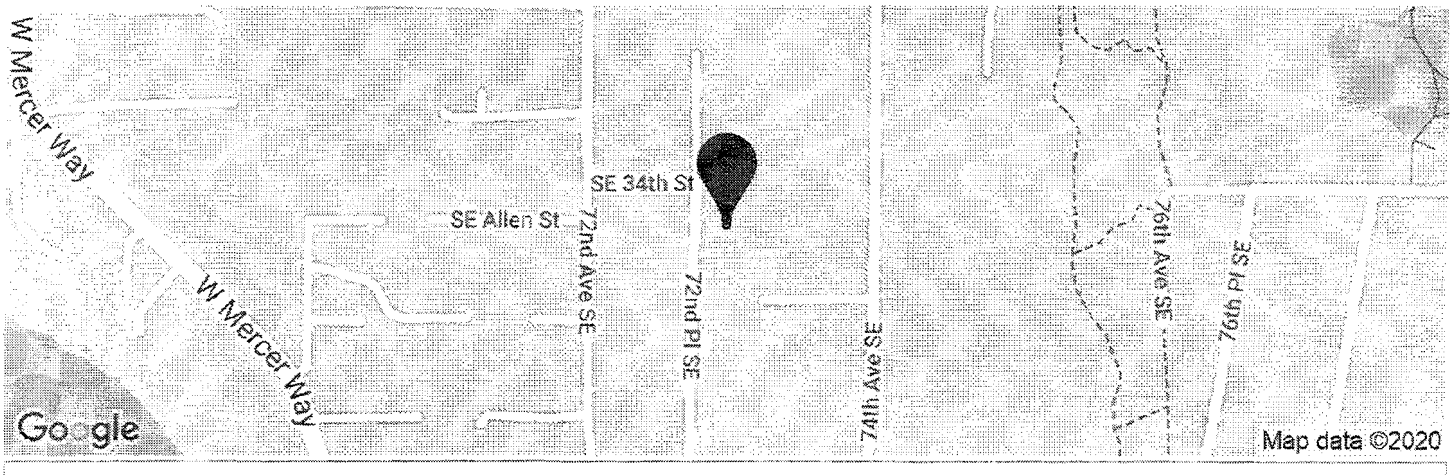
-FLOOR JOISTS & BEAMS OF EQUAL OR BETTER CAPACITY MAY BE SUBSTITUTED FOR THOSE SHOWN ON THIS PLAN, "EQUAL" IS DEFINED AS HAVING MOMENT CAPACITY, SHEAR CAPACITY, AND STIFFNESS WITHIN 3% OF THE SPECIFIED JOISTS OR BEAMS.

OSHPD

Lot 4 - Walia

3406 72nd Pl SE, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.5796977, -122.2418439



Date	7/17/2020, 10:19:54 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.395	MCE_R ground motion. (for 0.2 second period)
S_1	0.537	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.395	Site-modified spectral acceleration value
S_{M1}	0.805	Site-modified spectral acceleration value
S_{DS}	0.93	Numeric seismic design value at 0.2 second SA
S_{D1}	0.537	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	0.575	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.575	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.395	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.454	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.801	Factored deterministic acceleration value. (0.2 second)
S1RT	0.537	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.575	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.162	Factored deterministic acceleration value. (1.0 second)
PGAd	1.076	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.959	Mapped value of the risk coefficient at short periods

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)

$S_s := 1.395$ $S_1 := 0.537$ $S_{ms} := 1.395$ $S_{m1} := 0.805$

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: $S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{10}{12}\right)\right)} = 1.3$ $S_b := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{4}{12}\right)\right)} = 1.05$

Plan Area for Each Level:

$A_1 := 1560ft^2 \cdot S_a$ (Upper Roof) $A_{2a} := 1400ft^2$ (Framed Floor) $A_{2b} := 1905ft^2 \cdot S_b$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(40ft) + 2(36ft)$ (Upper Floor) $P_2 := 2(55ft) + 2(54ft)$ (Main Floor)

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

Weight of Structure at Each Level:

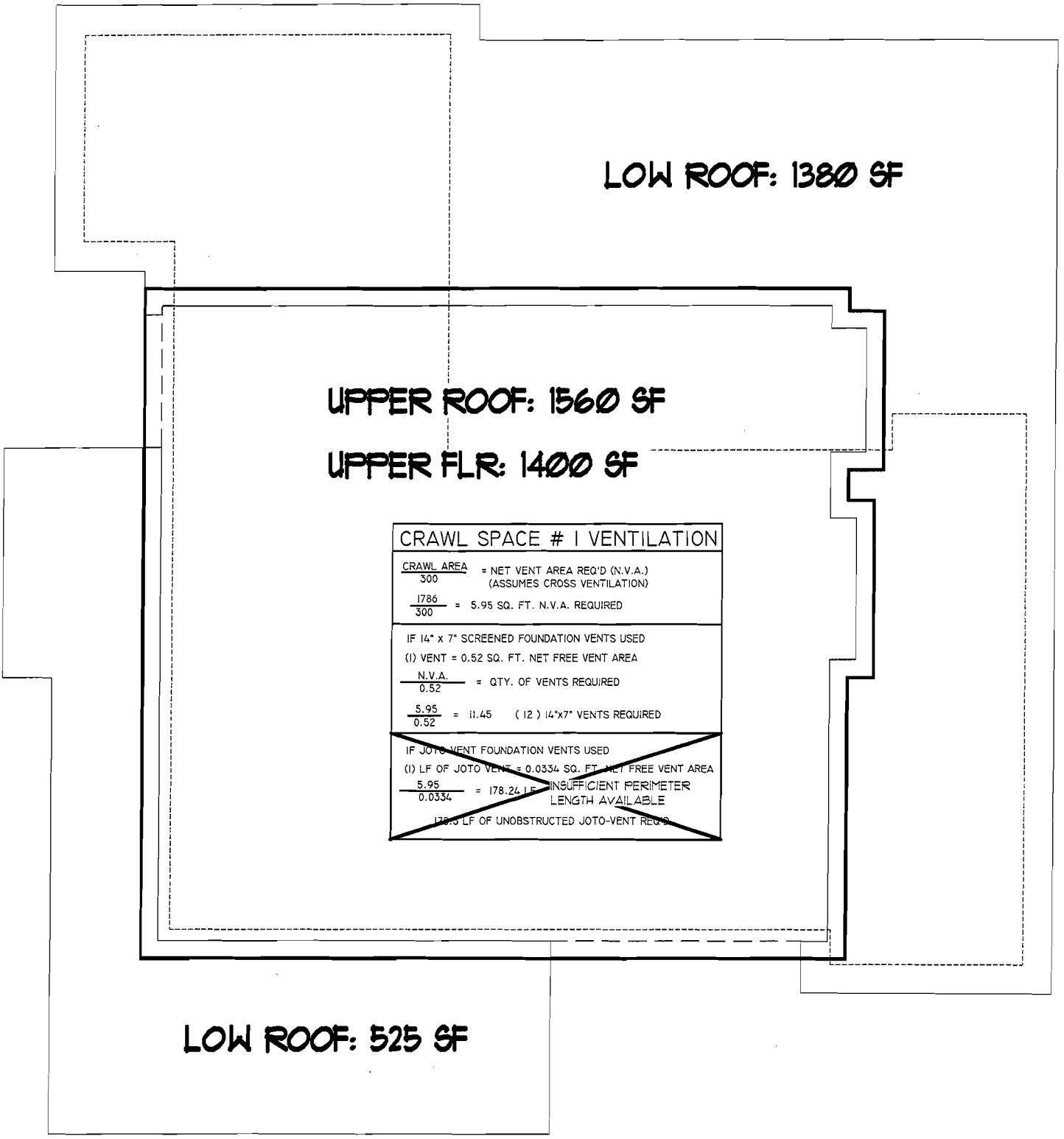
Story Weight at Upper Floor:

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

Story Weight at Main Floor:

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

$$W := w_1 + w_2 = 111076.67\,lb$$



CRAWL SPACE # 1 VENTILATION

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

$\frac{1786}{300}$ = 5.95 SQ. FT. N.V.A. REQUIRED

IF 14" x 7" SCREENED FOUNDATION VENTS USED

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

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

$\frac{5.95}{0.52}$ = 11.45 (12) 14"x7" VENTS REQUIRED

~~IF JOTO VENT FOUNDATION VENTS USED~~

~~(1) LF OF JOTO VENT = 0.0334 SQ. FT. NET FREE VENT AREA~~

~~$\frac{5.95}{0.0334}$ = 178.24 LF~~ ~~INSUFFICIENT PERIMETER LENGTH AVAILABLE~~

~~178.3 LF OF UNOBSTRUCTED JOTO-VENT REQ'D~~

Approximate Fundamental Period, T_a:

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

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

$C_s := \dfrac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.14$ Total Base Shear: $V_E := C_s \cdot W = 15892.51\,\text{lb}$

Vertical Shear distribution at each level:

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

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

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.52$ $F_1 := C_{v1} \cdot V_E = 8207.72\,\text{lb}$ Story Shear at Upper Floor

$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.48$ $F_2 := C_{v2} \cdot V_E = 7684.79\,\text{lb}$ Story Shear at Main Floor

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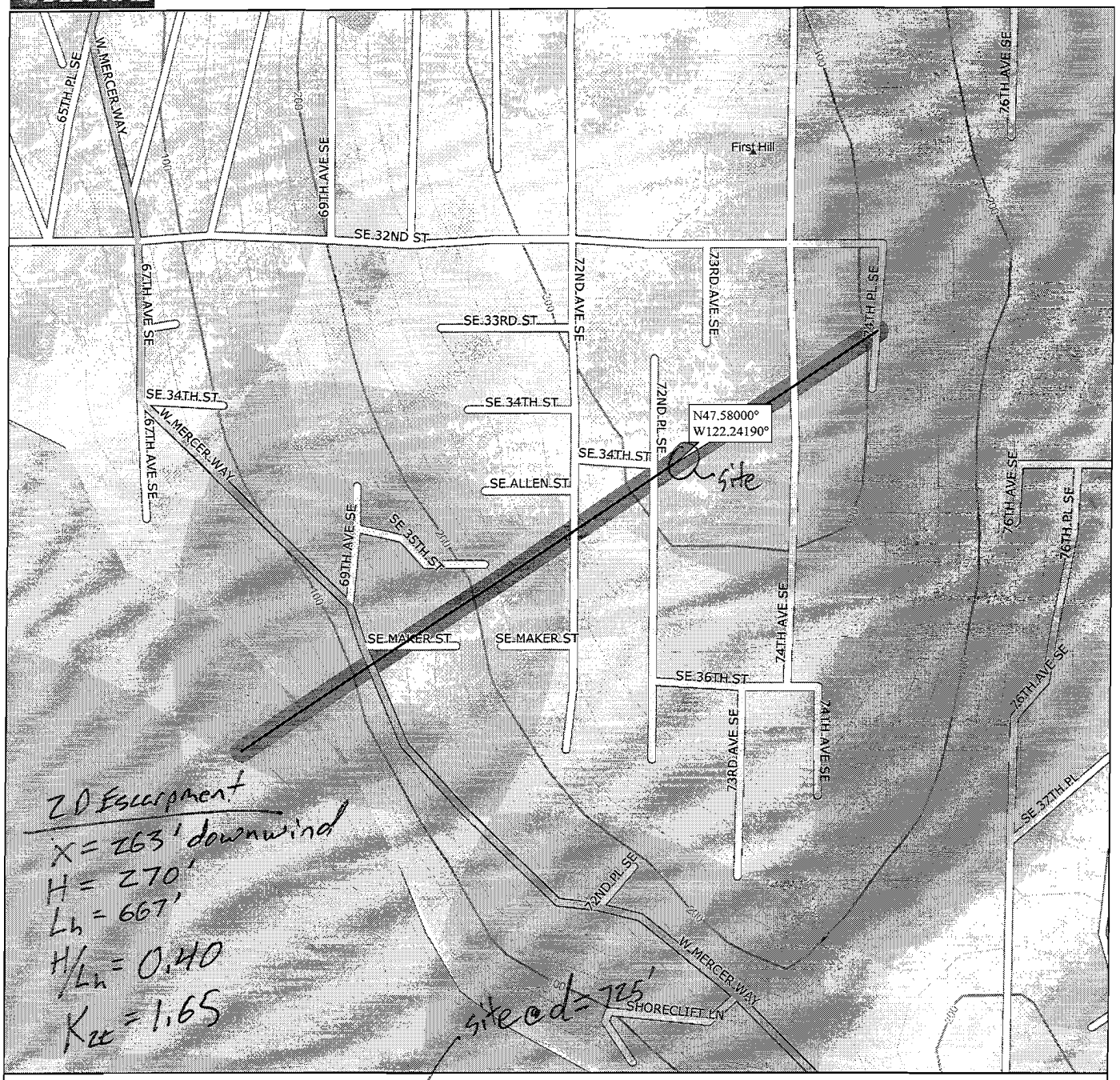

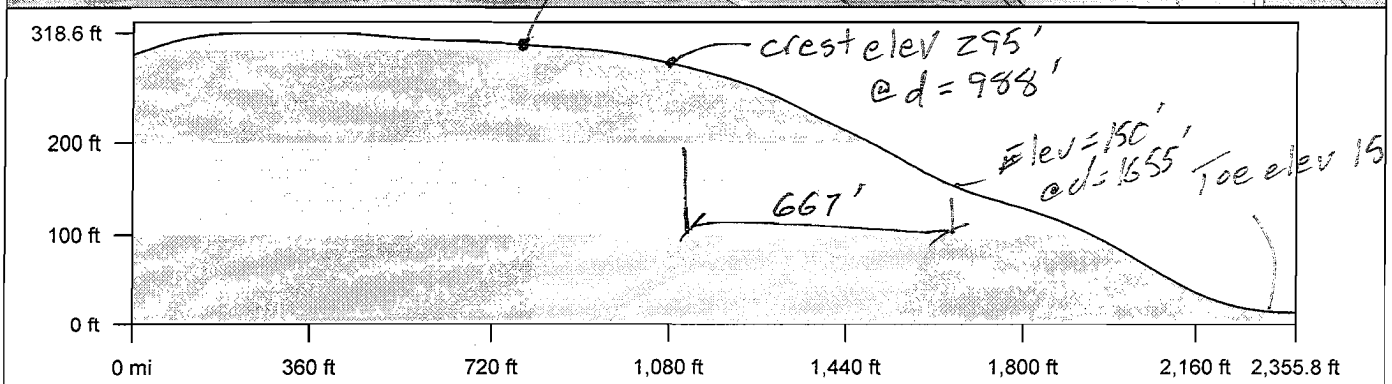

Lin Dist: 2,320.5 ft	Terr Dist: 2,355.8 ft	Elev Gain: -280.4 ft	Avg Grade: 14
Climb Elev: 24.0 ft	Desc Elev: 304.4 ft	Max. Elev: 318.6 ft	Min. Elev: 14.4 ft
Climb Dist: 308.1 ft	Desc Dist: 2,047.6 ft		

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TN
MN (15.2°E)

Scale 1 : 5,600
1" = 466.7 ft
Data Zoom 15-2

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 = 10/12 (40 degrees) Front to Back & 4.2/12 (20 degrees) Side to Side
Taken from Figure 27.4-1

Front to Back:

$L_{fb} := 54ft$ $B_{fb} := 55ft$ $\frac{L_{fb}}{B_{fb}} = 0.98$ $\frac{h}{L_{fb}} = 0.44$

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

$C_{pf2} := 0.36$ Windward Roof

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

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

Side to Side:

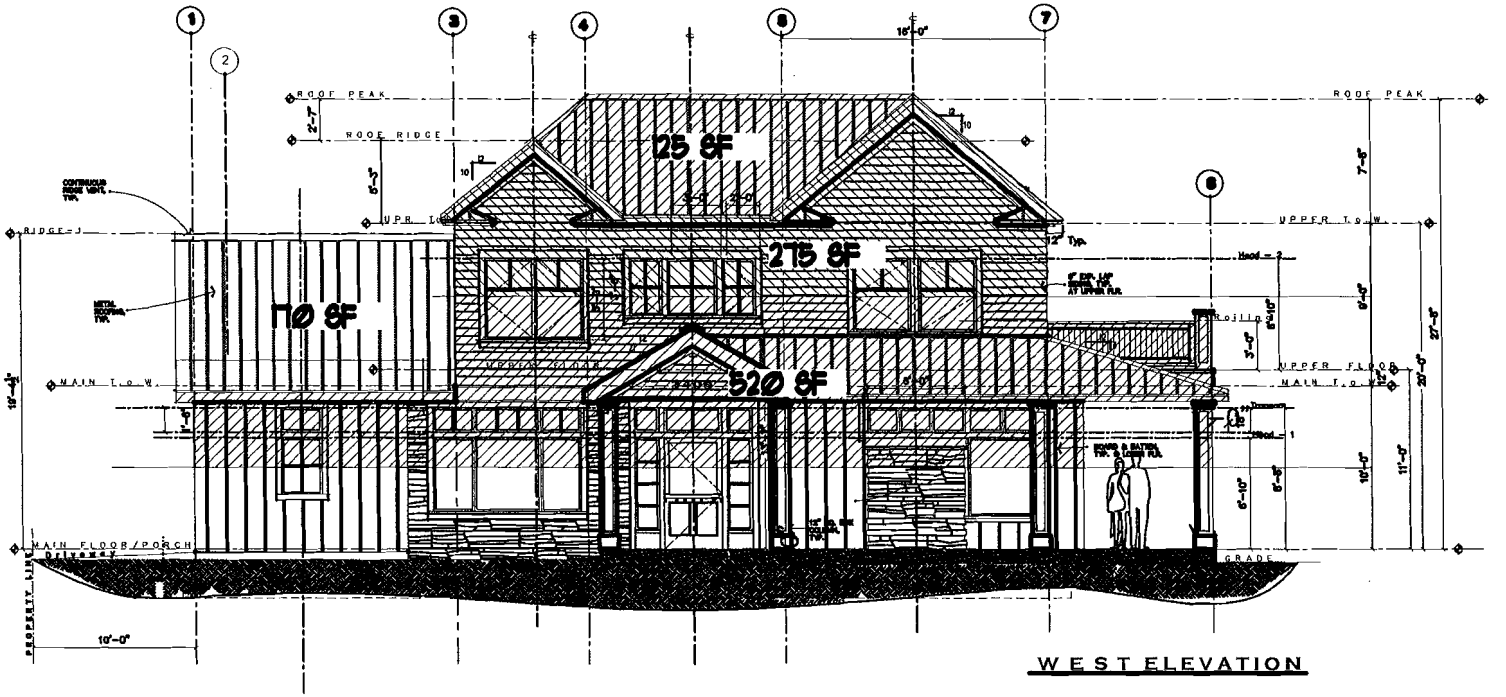
$L_{ss} := 55ft$ $B_{ss} := 54ft$ $\frac{L_{ss}}{B_{ss}} = 1.02$ $\frac{h}{L_{ss}} = 0.44$

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

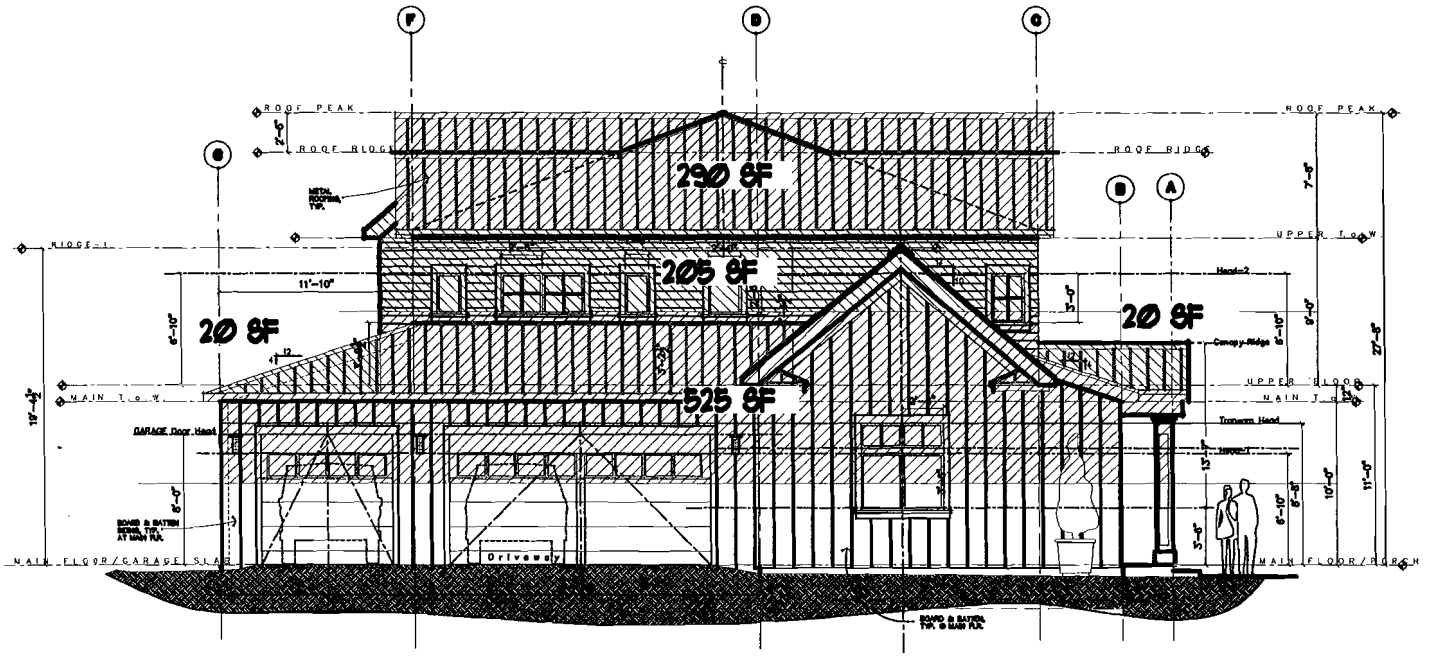
$C_{ps2} := 0.05$ Windward Roof

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

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



WEST ELEVATION



NORTH 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 $\quad p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 12.44\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $\quad p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -20.73\,ft^{-2} \cdot lb$

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 $\quad p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 1.73\,ft^{-2} \cdot lb$

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 $\quad p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -17.28\,ft^{-2} \cdot lb$

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:

$p_{wr1} - p_{lr1} = 33.17\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw1} = 43.88\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw1} = 42.31\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 22.46\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw2} = 43.88\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw2} = 42.31\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (N-S):

$V_{1W} := (p_{wr1} - p_{lr1})290ft^2 + (p_{ww1} - p_{lw1}) \cdot 205 \cdot ft^2 = 18613.48\,lb$

Wind Pressure at Main Floor (N-S):

$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 40ft^2 + (p_{ww1} - p_{lw1}) \cdot 525ft^2 = 24361.39\,lb$

Wind Pressure at Upper Roof (E-W):

$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 125ft^2 + (p_{ww1} - p_{lw2}) \cdot 275ft^2 = 14873.03\,lb$

Wind Pressure at Main Floor (N-S):

$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 170ft^2 + (p_{ww2} - p_{lw2}) \cdot 520ft^2 = 25820.28\,lb$

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)

$GC_{p1in} := 0.9$ $GC_{p2in} := 0.9$ $GC_{p3in} := 0.9$ Figure 30.4-2C (θ = 40 degrees)

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

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

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

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

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

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

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

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

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

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

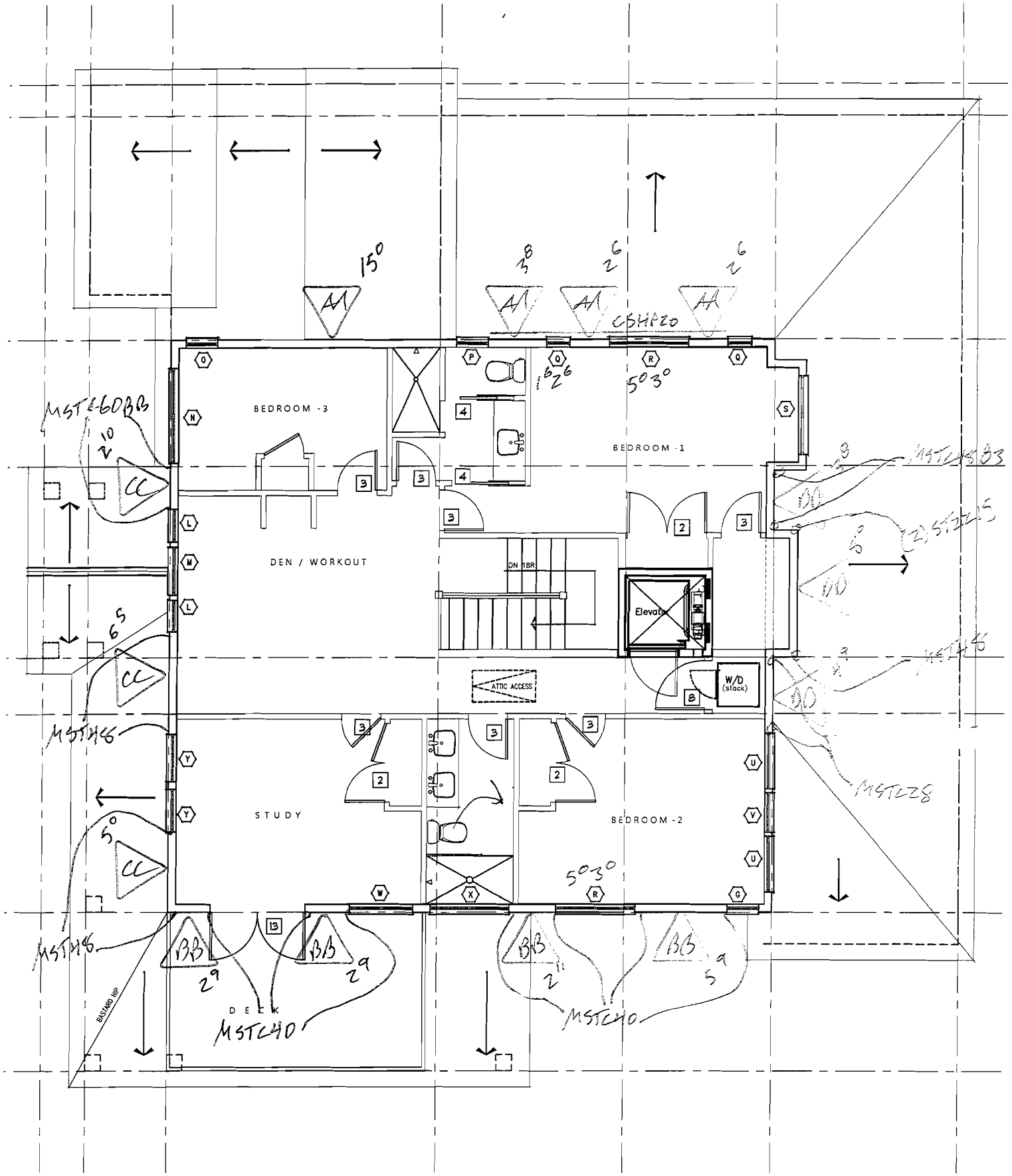
$p_5 := q_h \cdot [(GC_{p5out}) - (GC_{pi})]psf$ $p_5 = -64.22\,ft^{-2} \cdot lb$ (Zone 5)

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

$0.1(54ft) = 5.4\,ft$ $0.4 \cdot h = 9.6\,ft$ $0.04(46ft) = 1.84\,ft$

Therefore $a := 5.4ft$



North

UPPER FLOOR PLAN

AREA: 1,399 S.F.

11

WALL AA:

Story Shear due to Wind: $V_{3W} = 14873.03\,lb$

Story Shear due to Seismic: $F_1 = 8207.72\,lb$

Bldg Width in direction of Load: $L_t := 36 \cdot ft$

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

Shear Wall Length: $Laa_w := (15 + 3.67 + 2 \cdot 2.5)ft = 23.67\,ft$

$Laa_s := (15 + 3.67 + 2 \cdot 2.5)ft = 23.67\,ft$

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

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

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

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

$vaa = 188.5\,ft^{-1} \cdot lb \qquad \frac{vaa}{C_o} = 188.5\,ft^{-1} \cdot lb$

$E_{aa} = 121.36\,ft^{-1} \cdot lb \qquad \frac{E_{aa}}{C_o} = 121.36\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{aa} := 15 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{aa} := (15 \cdot psf) \cdot 6 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRaa := \frac{W_{aa} \cdot L_{aa}}{2} \qquad DLRaa = 1350\,lb$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} \qquad CFaa_w = 1696.54\,lb$

$CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} \qquad CFaa_s = 1092.28\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 886.54\,lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 458.05\,lb$

No Holdown Required

Dead Load Resisting Overturning: $L_{aa} := 15.17 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{aa1} := (15 \cdot psf) \cdot 6 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRaa1 := \frac{W_{aa1} \cdot L_{aa}}{2}$ $DLRaa = 1350\,lb$

Chord Force:

$CFaa_{w1} := \frac{vaa \cdot 8.67ft \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_{w1} = 969.61\,lb$

$CFaa_{s1} := \frac{E_{aa} \cdot 8.67ft \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_{s1} = 624.26\,lb$

Holdown Force:

$HDFaa_{w1} := CFaa_{w1} - 0.6 \cdot DLRaa1 = 150.43\,lb$

$HDFaa_{s1} := CFaa_{s1} - (0.6 - 0.14S_{DS})DLRaa1 = -17.15\,lb$

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

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

$B_p := \frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 0.87\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 1.34\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vaa} = 7.3\,ft$ $\frac{(Z_B \cdot C_o)}{E_{aa}} = 11.34\,ft$

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

WALL BB:

Story Shear due to Wind: $V_{3W} = 14873.03\,lb$

Story Shear due to Seismic: $F_1 = 8207.72\,lb$

Bldg Width in direction of Load: $L_t := 36 \cdot ft$

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

Shear Wall Length:

$Lbb_w := (2 \cdot 2.75 + 2.92 + 5.75)ft = 14.17\,ft$

$$Lbb_s := \left[2 \cdot 2.75\left(\frac{5.5}{9}\right) + 2.92\left(\frac{5.83}{9}\right) + 5.75\right]ft = 11\,ft$$

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

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

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

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

$vbb = 314.88\,ft^{-1} \cdot lb$ $\quad \frac{vbb}{C_o} = 314.88\,ft^{-1} \cdot lb$

$E_{bb} = 261.09\,ft^{-1} \cdot lb$ $\quad \frac{E_{bb}}{C_o} = 261.09\,ft^{-1} \cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 364 plf
Seismic Capacity = 260 plf (OK, within 1%)

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

$W_{bb} := (15 \cdot psf) \cdot 7.5 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2} \qquad DLRbb = 278.44\,lb$$

Chord Force:

$$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} \qquad CFbb_w = 2833.96\,lb$$

$$CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} \qquad CFbb_s = 2349.83\,lb$$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 2666.9\,lb$

$HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS}) \cdot DLRbb = 2219.02\,lb$

Simpson MSTC40

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

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

$$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 0.52\,ft \qquad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.63\,ft$$

16d @ 6" o.c.

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

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

$$A_s := \frac{(Z_B \cdot C_o)}{vbb} = 4.37\,ft \qquad \frac{(Z_B \cdot C_o)}{E_{bb}} = 5.27\,ft$$

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 18613.48\,lb$

Story Shear due to Seismic: $F_1 = 8207.72\,lb$

Bldg Width in direction of Load: $L_t := 40 \cdot ft$

Distance between shear walls: $L_1 := 40 \cdot ft$

Shear Wall Length: $Lcc_w := (5 + 6.42 + 2.83)ft = 14.25\,ft$

$$Lcc_s := \left[5 + 6.42 + 2.83\left(\frac{5.67}{9}\right)\right]ft = 13.2\,ft$$

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

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

Wind Force: $$vcc := \frac{\frac{0.6V_{1W}}{L_t} \cdot \frac{L_1}{2}}{Lcc_w}$$

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

$vcc = 391.86\,ft^{-1} \cdot lb$ $\quad \frac{vcc}{C_o} = 391.86\,ft^{-1} \cdot lb$

$E_{cc} = 217.58\,ft^{-1} \cdot lb$ $\quad \frac{E_{cc}}{C_o} = 217.58\,ft^{-1} \cdot lb$

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

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

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

$$DLRcc := \frac{W_{cc} \cdot L_{cc}}{2} \qquad DLRcc = 169.8\,lb$$

Chord Force:

$$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} \qquad CFcc_w = 3526.77\,lb$$

$$CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} \qquad CFcc_s = 1958.23\,lb$$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 3424.89\,lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 1878.46\,lb$

Simpson MST48 to wall below, or MSTC60B3 to Header

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

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

$$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 0.42\,ft \qquad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 0.75\,ft$$

16d @ 4" o.c.

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

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

$$A_s := \frac{(Z_B \cdot C_o)}{vcc} = 3.51\,ft \qquad \frac{(Z_B \cdot C_o)}{E_{cc}} = 6.32\,ft$$

5/8" A.B. @ 42" o.c.

WALL DD:

Story Shear due to Wind: $V_{1W} = 18613.48\,lb$

Story Shear due to Seismic: $F_1 = 8207.72\,lb$

Bldg Width in direction of Load: $L_t := 40 \cdot ft$

Distance between shear walls: $L_1 := 40 \cdot ft$

Shear Wall Length: $Ldd_w := (3.67 + 4.75 + 8)ft = 16.42\,ft$

$$Ldd_s := \left[3.67\left(\frac{7.33}{9}\right) + 4.75 + 8\right]ft = 15.74\,ft$$

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

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

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

Seismic Force: $\rho := 1.0$

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

$vdd = 340.08\,ft^{-1} \cdot lb \qquad \frac{vdd}{C_o} = 340.08\,ft^{-1} \cdot lb$

$E_{dd} = 182.52\,ft^{-1} \cdot lb \qquad \frac{E_{dd}}{C_o} = 182.52\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 3.67 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$$DLRdd := \frac{W_{dd} \cdot L_{dd}}{2} \qquad DLRdd = 220.2\,lb$$

Chord Force:

$$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} \qquad CFdd_w = 3060.68\,lb$$

$$CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} \qquad CFdd_s = 1642.69\,lb$$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 2928.56\,lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 1539.24\,lb$

Simpson MSTC48B3 to beam or MST48 to wall or (2) ST2215 to flush beam
OR MSTC24

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

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

$$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 0.48\,ft \qquad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 0.89\,ft$$

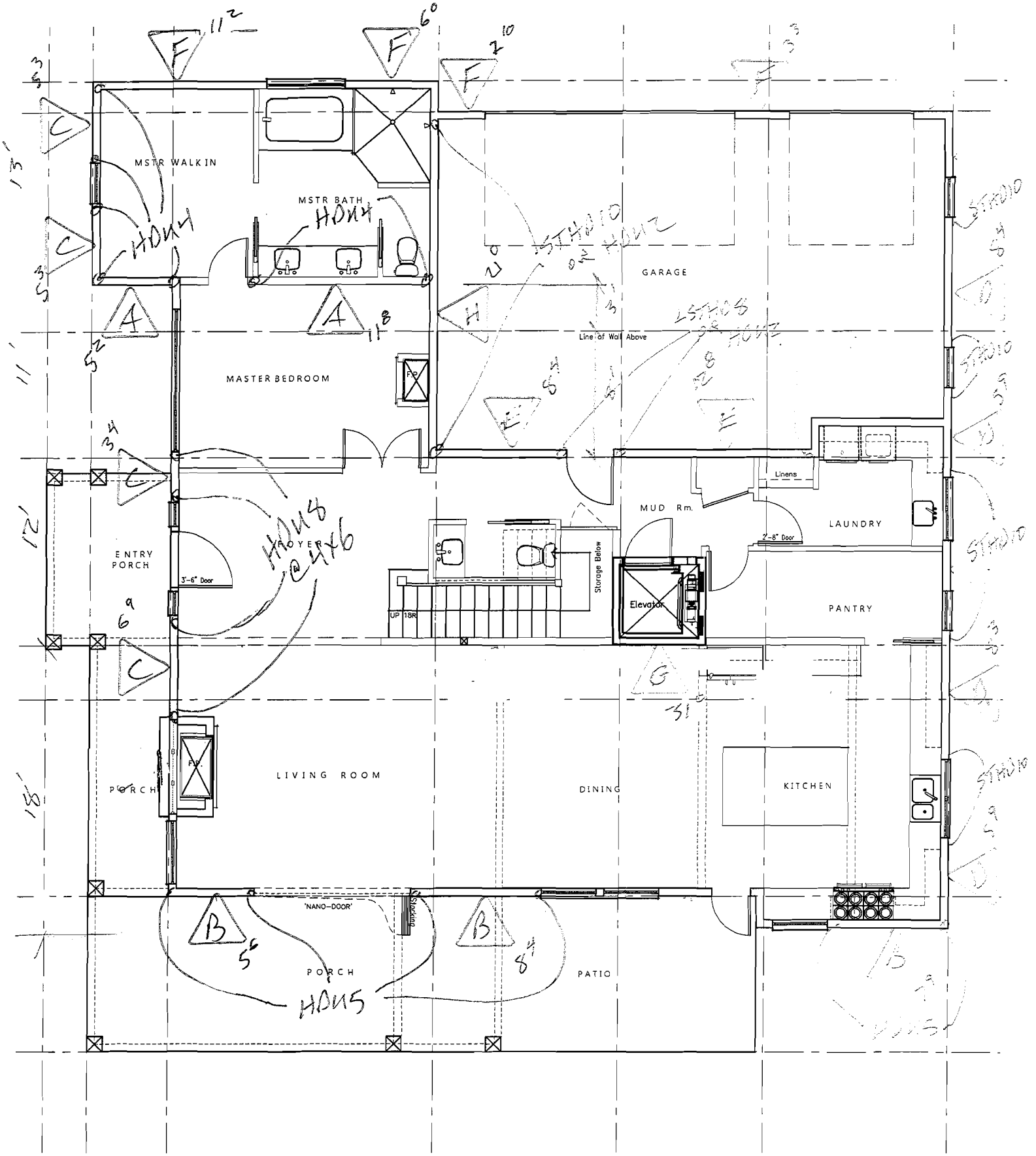
16d @ 6" o.c.

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

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

$$A_s := \frac{(Z_B \cdot C_o)}{vdd} = 4.05\,ft \qquad \frac{(Z_B \cdot C_o)}{E_{dd}} = 7.54\,ft$$

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



North

MAIN FLOOR PLAN

HOUSE AREA : 1,877.5 S.F.
GARAGE AREA : 719 S.F.

WALL A:

Story Shear due to Wind: $V_{4W} = 25820.28\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 13 \cdot ft$ $L_2 := 11ft$

Shear Wall Length: $La_w := (5.17 + 11.67)ft = 16.84\,ft$

$La_s := (5.17 + 11.67)ft = 16.84\,ft$

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: $va := \dfrac{\frac{8}{11} \cdot (vaa \cdot La_w) + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{La_w}$

Seismic Force: $\rho := 1.0$ $E_a := \dfrac{\frac{8}{11} \cdot (E_{aa} \cdot La_s) + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{La_s}$

$va = 397.13\,ft^{-1} \cdot lb$ $\frac{va}{C_o} = 397.13\,ft^{-1} \cdot lb$

$E_a = 195.05\,ft^{-1} \cdot lb$ $\frac{E_a}{C_o} = 195.05\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_a := 5.17 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRa := \frac{W_a \cdot L_a}{2}$ $DLRa = 336.05\,lb$

Chord Force:

$CFa_w := \frac{va \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_w = 3971.34\,lb$

$CFa_s := \frac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_s = 1950.5\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 3769.71\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS}) \cdot DLRa = 1792.63\,lb$

Simpson HDU4 w/ SSTB20 anchor or PAB5 anchor (6" embed)

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{va} = 0.41\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.84\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{va} = 3.46\,ft$ $\frac{(Z_B \cdot C_o)}{E_a} = 7.05\,ft$

5/8" A.B. @ 42" o.c.

WALL B:

Story Shear due to Wind: $V_{4W} = 25820.28\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 18 \cdot ft$

Shear Wall Length:

$Lb_w := (5.5 + 8.33 + 7.75)ft = 21.58\,ft$

$Lb_s := (5.5 + 8.33 + 7.75)ft = 21.58\,ft$

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

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

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

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

$vb = 326.41\,ft^{-1} \cdot lb \qquad \dfrac{vb}{C_o} = 326.41\,ft^{-1} \cdot lb$

$E_b = 174.66\,ft^{-1} \cdot lb \qquad \dfrac{E_b}{C_o} = 174.66\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_b := 5.5 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRb := \dfrac{W_b \cdot L_b}{2} \qquad DLRb = 495\,lb$

Chord Force:

$CFb_w := \dfrac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b} \qquad CFb_w = 3264.1\,lb$

$CFb_w + CFbb_w = 6098.06\,lb$

$CFb_s := \dfrac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b} \qquad CFb_s = 1746.65\,lb$

$CFb_s + CFbb_s = 4096.48\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 2967.1\,lb$

$HDFb_w + HDFbb_w = 5634\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14S_{DS}) \cdot DLRb = 1514.09\,lb$

$HDFb_s + HDFbb_s = 3733.12\,lb$

Simpson HDU5 w/ SB5/8x24 anchor

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

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

$B_p := \dfrac{(C_D \cdot Z_N \cdot C_o)}{vb} = 0.5\,ft \qquad \dfrac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.93\,ft$

16d @ 6" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vb} = 4.22\,ft \qquad \dfrac{(Z_B \cdot C_o)}{E_b} = 7.88\,ft$

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

WALL C:

Story Shear due to Wind: $V_{2W} = 24361.39\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

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

Distance between shear walls: $L_l := 22 \cdot ft$

Shear Wall Length: $Lc_w := (2 \cdot 5.25 + 6.75 + 3.33)ft = 20.58\,ft$

$Lc_s := \left[2 \cdot 5.25 + 6.75 + 3.33\left(\frac{6.67}{10}\right)\right]ft = 19.47\,ft$

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

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

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

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

$vc = 413.38\,ft^{-1} \cdot lb$ $\quad \frac{vc}{C_o} = 413.38\,ft^{-1} \cdot lb$

$E_c = 202.79\,ft^{-1} \cdot lb$ $\quad \frac{E_c}{C_o} = 202.79\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_c := 3.33 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_c := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1\,ft$

$DLRc := \frac{W_c \cdot L_c}{2}$ $\quad DLRc = 233.1\,lb$

Chord Force:

$CFc_w := \frac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $\quad CFc_w = 4133.83\,lb$

$CFc_w + CFcc_w = 7660.59\,lb$

$CFc_s := \frac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $\quad CFc_s = 2027.91\,lb$

$CFc_s + CFcc_s = 3986.14\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 3993.97\,lb$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS}) \cdot DLRc = 1918.4\,lb$

Simpson STHD10 or HDU4 w/ SSTB20 anchor

$HDFc_w + HDFcc_w = 7418.85\,lb$

$HDFc_s + HDFcc_s = 3796.86\,lb$

Simpson HDU8 at 4x6 post 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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vc} = 0.39\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 0.8\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vc} = 3.33\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_c} = 6.79\,ft$

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

WALL D:

Story Shear due to Wind: $V_{2W} = 24361.39\,lb$ Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$ Distance between shear walls: $L_1 := 33 \cdot ft$

Shear Wall Length: $Ld_w := (2 \cdot 5.75 + 8.25 + 8.33)ft = 28.08\,ft$ $Ld_s := (2 \cdot 5.75 + 8.25 + 8.33)ft = 28.08\,ft$

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

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

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

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

$vd = 355.02\,ft^{-1} \cdot lb$ $\frac{vd}{C_o} = 355.02\,ft^{-1} \cdot lb$

$E_d = 159.78\,ft^{-1} \cdot lb$ $\frac{E_d}{C_o} = 159.78\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_d := 5.75 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_d := (15 \cdot psf) \cdot 6 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 546.25\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 3550.25\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 1597.76\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 3222.5\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 1341.13\,lb$

Simpson STHD10 OR HDU4

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 0.46\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 1.02\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vd} = 3.88\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 8.61\,ft$

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

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WALL E:

Story Shear due to Wind: $V_{4W} = 25820.28\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 12 \cdot ft$ $L_2 := 11\,ft$

Shear Wall Length: $Le_w := (8.33 + 12.67)ft = 21\,ft$

$Le_s := (8.33 + 12.67)ft = 21\,ft$

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: $ve := \dfrac{\frac{3}{11} \cdot (vaa \cdot Laa_w) + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le_w}$

Seismic Force: $\rho := 1.0$ $E_e := \dfrac{\frac{3}{11} \cdot (E_{aa} \cdot Laa_s) + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le_s}$

$ve = 215.05\,ft^{-1} \cdot lb$ $\frac{ve}{C_o} = 215.05\,ft^{-1} \cdot lb$

$E_e = 91.86\,ft^{-1} \cdot lb$ $\frac{E_e}{C_o} = 91.86\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_e := 8.33 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_e := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 6ft$

$DLRe := \frac{W_e \cdot L_e}{2}$ $DLRe = 666.4\,lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_w = 2150.54\,lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_s = 918.6\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 1750.7\,lb$

$HDFe_s := CFe_s - (0.6 - 0.14S_{DS}) \cdot DLRe = 605.53\,lb$

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{ve} = 0.76\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 1.78\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{ve} = 6.4\,ft$ $\frac{(Z_B \cdot C_o)}{E_e} = 14.98\,ft$

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

WALL F:

Story Shear due to Wind: $V_{4W} = 25820.28\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

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

Shear Wall Length:

$Lf_w := (11.17 + 6 + 2.83 + 3.25)ft = 23.25\,ft$

$Lf_s := \left[11.17 + 6 + 2.83\left(\frac{5.67}{10}\right) + 3.25\left(\frac{6.5}{10}\right)\right]ft = 20.89\,ft$

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

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

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

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

$vf = 80.21\,ft^{-1} \cdot lb$ $\quad \frac{vf}{C_o} = 80.21\,ft^{-1} \cdot lb$

$E_f = 31\,ft^{-1} \cdot lb$ $\quad \frac{E_f}{C_o} = 31\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_f := 2.83 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRf := \frac{W_f \cdot L_f}{2}$ $\quad DLRf = 311.3\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $\quad CFf_w = 802.06\,lb$

$CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $\quad CFf_s = 310.01\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 615.28\,lb$

$HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 163.76\,lb$

No Holdown Required

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

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

$B_p := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 2.03\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 5.26\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vf} = 17.16\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_f} = 44.39\,ft$

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

WALL G:

Story Shear due to Wind: $V_{4W} = 25820.28\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 12 \cdot ft$ $L_2 := 18ft$

Shear Wall Length: $Lg_w := (31)ft = 31\,ft$

$Lg_s := (31)ft = 31\,ft$

Percent full height sheathing: $\% := \left(\frac{31 \cdot ft}{31 \cdot ft}\right) \cdot 100$ $\% = 100$

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

Wind Force: $vg := \frac{\left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg_w}$

Seismic Force: $\rho := 1.0$ $E_g := \frac{\left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg_s}$

$vg = 138.82\,ft^{-1} \cdot lb$ $\frac{vg}{C_o} = 138.82\,ft^{-1} \cdot lb$

$E_g = 48.2\,ft^{-1} \cdot lb$ $\frac{E_g}{C_o} = 48.2\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_g := 31 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRg := \frac{W_g \cdot L_g}{2}$ $DLRg = 2790\,lb$

Chord Force:

$CFg_w := \frac{vg \cdot L_g \cdot Pt}{C_o \cdot L_g}$ $CFg_w = 1388.19\,lb$

$CFg_s := \frac{E_g \cdot L_g \cdot Pt}{C_o \cdot L_g}$ $CFg_s = 482.02\,lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6 \cdot DLRg = -285.81\,lb$

$HDFg_s := CFg_s - (0.6 - 0.14S_{DS}) \cdot DLRg = -828.72\,lb$

No Holdown Rquired

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vg} = 1.18\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_g} = 3.39\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vg} = 9.91\,ft$ $\frac{(Z_B \cdot C_o)}{E_g} = 28.55\,ft$

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

WALL H:

Story Shear due to Wind: $V_{2W} = 24361.39\,lb$

Story Shear due to Seismic: $F_2 = 7684.79\,lb$

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

Distance between shear walls: $L_1 := 22 \cdot ft$ $L_2 := 33ft$

Shear Wall Length: $Lh_w := (21)ft = 21\,ft$

$Lh_s := (21)ft = 21\,ft$

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

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

Wind Force: $vh := \frac{\left(\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lh_w}$

Seismic Force: $\rho := 1.0$ $E_h := \frac{\left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lh_s}$

$vh = 348.02\,ft^{-1} \cdot lb$ $\frac{vh}{C_o} = 348.02\,ft^{-1} \cdot lb$

$E_h = 128.08\,ft^{-1} \cdot lb$ $\frac{E_h}{C_o} = 128.08\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_h := 21 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRh := \frac{W_h \cdot L_h}{2}$ $DLRh = 1260\,lb$

Chord Force:

$CFh_w := \frac{vh \cdot L_h \cdot Pt}{C_o \cdot L_h}$ $CFh_w = 3480.2\,lb$

$CFh_s := \frac{E_h \cdot L_h \cdot Pt}{C_o \cdot L_h}$ $CFh_s = 1280.8\,lb$

Holdown Force:

$HDFh_w := CFh_w - 0.6 \cdot DLRh = 2724.2\,lb$

$HDFh_s := CFh_s - (0.6 - 0.14S_{DS}) \cdot DLRh = 688.85\,lb$

Simpson STHD10 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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vh} = 0.47\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_h} = 1.27\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vh} = 3.95\,ft$ $\frac{(Z_B \cdot C_o)}{E_h} = 10.74\,ft$

5/8" A.B. @ 48" 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:

$$vaa \cdot \frac{Laa_w}{38ft} = 117.42\,ft^{-1} \cdot lb \qquad E_{aa} \cdot \frac{Laa_s}{38ft} = 75.6\,ft^{-1} \cdot lb$$

Wall Lines BB:

$$vbb \cdot \frac{Lbb_w}{38ft} = 117.42\,ft^{-1} \cdot lb \qquad E_{bb} \cdot \frac{Lbb_s}{38ft} = 75.6\,ft^{-1} \cdot lb$$

Wall Lines CC:

$$vcc \cdot \frac{Lcc_w}{36ft} = 155.11\,ft^{-1} \cdot lb \qquad E_{cc} \cdot \frac{Lcc_s}{36ft} = 79.8\,ft^{-1} \cdot lb$$

Wall Lines DD:

$$vdd \cdot \frac{Ldd_w}{36ft} = 155.11\,ft^{-1} \cdot lb \qquad E_{dd} \cdot \frac{Ldd_s}{36ft} = 79.8\,ft^{-1} \cdot lb$$

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:

$$\frac{va \cdot La_w}{55ft} = 121.6\,ft^{-1} \cdot lb \qquad \frac{E_a \cdot La_s}{55ft} = 59.72\,ft^{-1} \cdot lb$$

Wall Lines B:

$$\frac{vb \cdot Lb_w - vbb \cdot Lbb_w}{50ft} = 51.64\,ft^{-1} \cdot lb \qquad \frac{E_b \cdot Lb_s - E_{bb} \cdot Lbb_s}{50ft} = 17.93\,ft^{-1} \cdot lb \qquad \frac{vb \cdot Lb_w}{50ft} = 140.88\,ft^{-1} \cdot lb \qquad \frac{E_b \cdot Lb_s}{50ft} = 75.39\,ft^{-1} \cdot lb$$

Wall Lines C:

$$\frac{vc \cdot Lc_w - vcc \cdot Lcc_w}{52ft} = 56.22\,ft^{-1} \cdot lb \qquad \frac{E_c \cdot Lc_s - E_{cc} \cdot Lcc_s}{52ft} = 20.69\,ft^{-1} \cdot lb \qquad \frac{vc \cdot Lc_w}{52ft} = 163.6\,ft^{-1} \cdot lb \qquad \frac{E_c \cdot Lc_s}{52ft} = 75.93\,ft^{-1} \cdot lb$$

Wall Lines D:

$$\frac{vd \cdot Ld_w - vdd \cdot Ldd_w}{52ft} = 84.33\,ft^{-1} \cdot lb \qquad \frac{E_d \cdot Ld_s - E_{dd} \cdot Ldd_s}{52ft} = 31.03\,ft^{-1} \cdot lb \qquad \frac{vd \cdot Ld_w}{52ft} = 191.71\,ft^{-1} \cdot lb \qquad \frac{E_d \cdot Ld_s}{52ft} = 86.28\,ft^{-1} \cdot lb$$

Wall Lines E:

$$ve \cdot \frac{Le_w}{50ft} = 90.32\,ft^{-1} \cdot lb \qquad E_e \cdot \frac{Le_s}{50ft} = 38.58\,ft^{-1} \cdot lb$$

Wall Lines F:

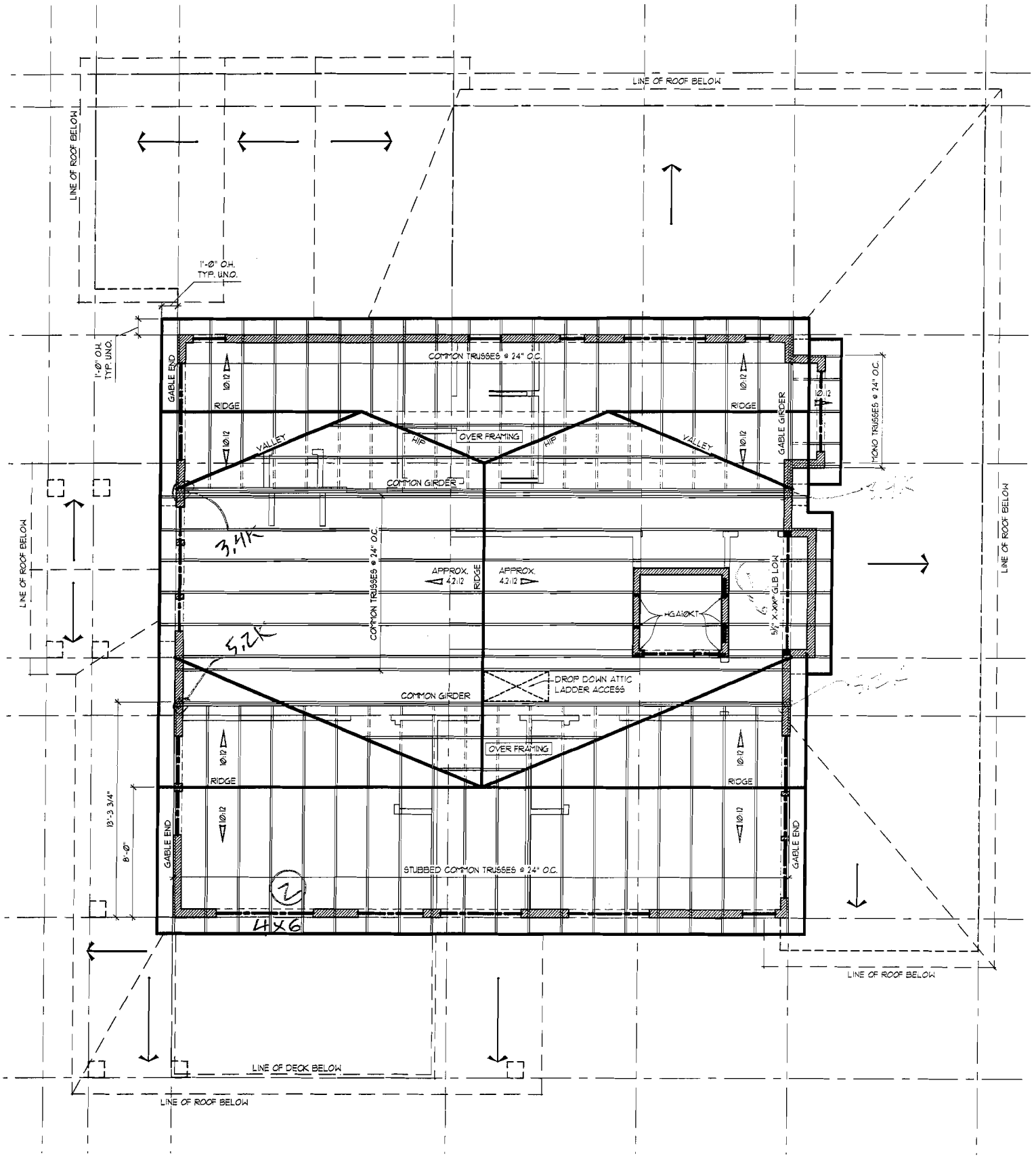
$$vf \cdot \frac{Lf_w}{55ft} = 33.91\,ft^{-1} \cdot lb \qquad E_f \cdot \frac{Lf_s}{55ft} = 11.77\,ft^{-1} \cdot lb$$

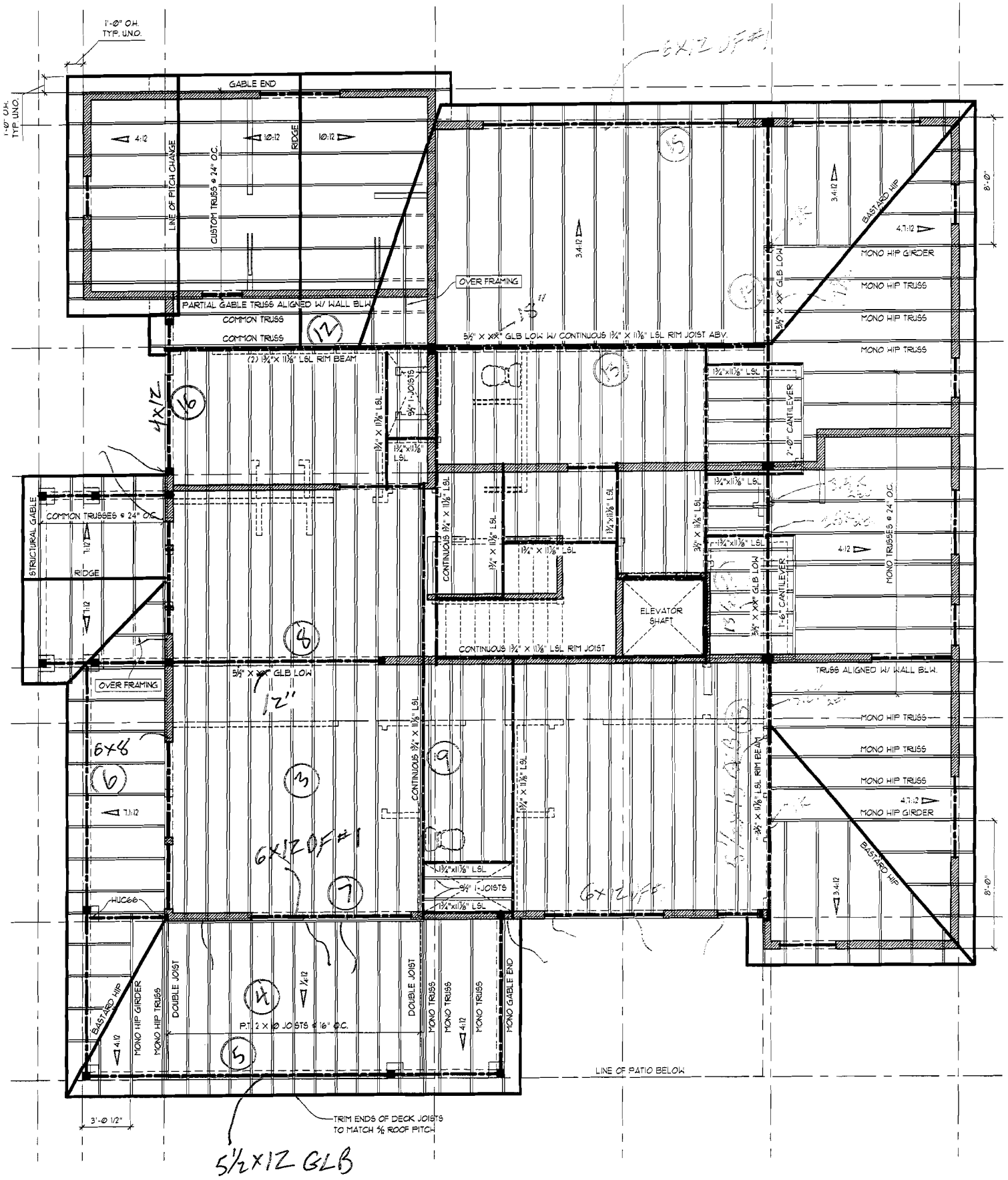
Wall Lines G:

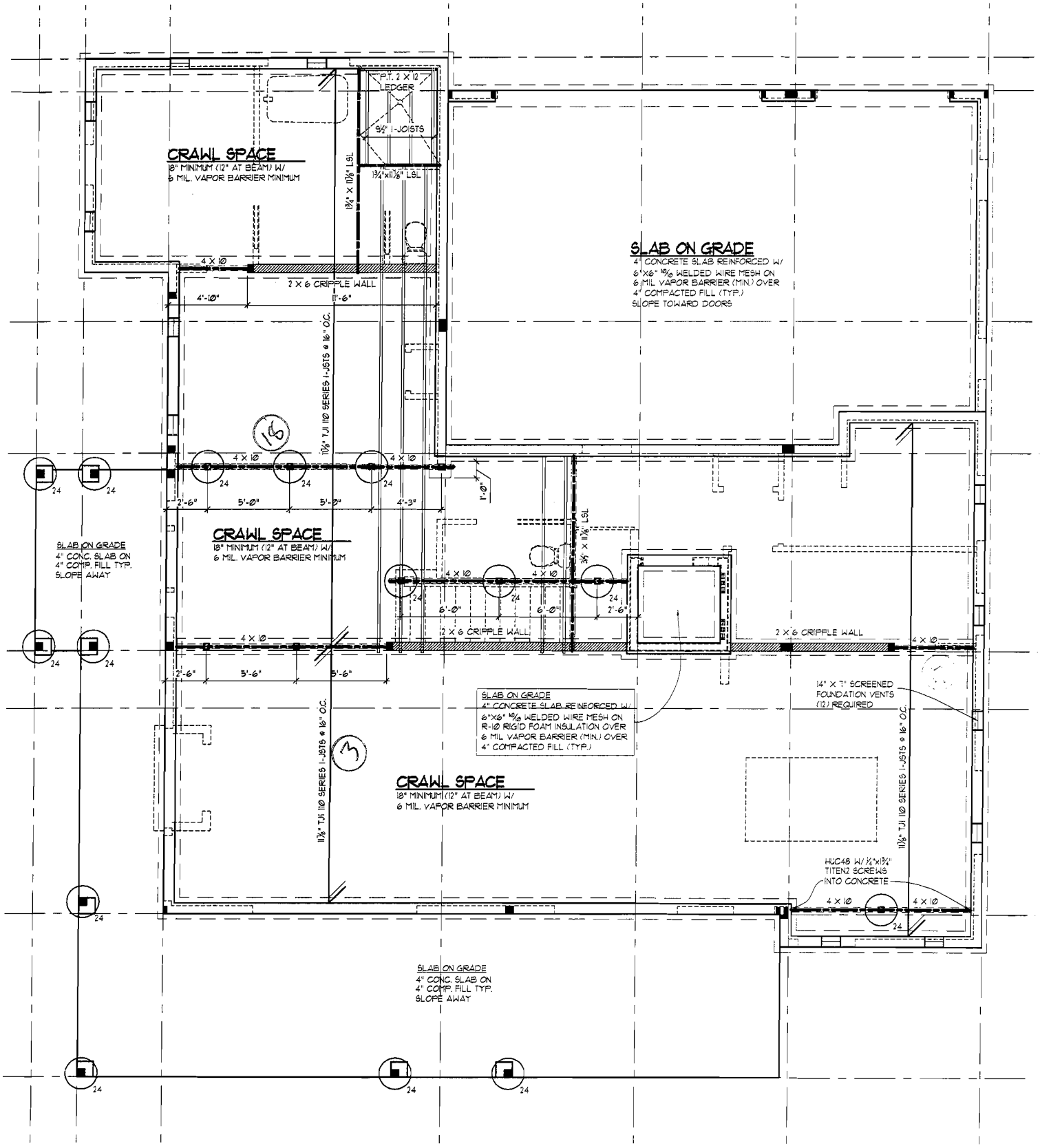
$$vg \cdot \frac{Lg_w}{50ft} = 86.07\,ft^{-1} \cdot lb \qquad E_g \cdot \frac{Lg_s}{50ft} = 29.89\,ft^{-1} \cdot lb$$

Wall Lines H:

$$vh \cdot \frac{Lh_w}{52ft} = 140.55\,ft^{-1} \cdot lb \qquad E_h \cdot \frac{Lh_s}{52ft} = 51.72\,ft^{-1} \cdot lb$$


LINE OF ROOF BELOW
LINE OF ROOF BELOW
1'-0" O.H. TYP. U.N.O.
1'-0" O.H. TYP. U.N.O.
GABLE END
COMMON TRUSSES @ 24" O.C.
10:12
RIDGE
VALLEY
HIP
OVER FRAMING
HIP
VALLEY
RIDGE
GABLE GIRDER
MONO TRUSSES @ 24" O.C.
COMMON GIRDER
3.4K
COMMON TRUSSES @ 24" O.C.
APPROX. 4:12
RIDGE
APPROX. 4:12
HG A10KT
5½" X XXX" GLB LOW
5.2K
DROP DOWN ATTIC LADDER ACCESS
COMMON GIRDER
OVER FRAMING
RIDGE
RIDGE
13'-3 3/4"
8'-0"
GABLE END
GABLE END
STUBBED COMMON TRUSSES @ 24" O.C.
2
4X6
LINE OF ROOF BELOW
LINE OF ROOF BELOW
LINE OF ROOF BELOW
LINE OF DECK BELOW
LINE OF ROOF BELOW


1'-0" O.H. TYP. U.N.O.
GABLE END
4:12
10:12
RIDGE
LINE OF PITCH CHANGE
CUSTOM TRUSS @ 24" O.C.
PARTIAL GABLE TRUSS ALIGNED W/ WALL BLW.
COMMON TRUSS
COMMON TRUSS
OVER FRAMING
5½" X 18" GLB LOW W/ CONTINUOUS 1¾" X 11⅞" LSL RIM JOIST ABV.
(2) 1¾"X 11⅞" LSL RIM BEAM
1¾" X 11⅞" LSL
9½" I-JOISTS
1¾"x11⅞" LSL
STRUCTURAL GABLE
COMMON TRUSSES @ 24" O.C.
7:12
RIDGE
CONTINUOUS 1¾" X 11⅞" LSL
ELEVATOR SHAFT
CONTINUOUS 1¾" X 11⅞" LSL RIM JOIST
5½" X 18" GLB LOW
OVER FRAMING
TRUSS ALIGNED W/ WALL BLW.
MONO HIP GIRDER
MONO HIP TRUSS
MONO HIP TRUSS
MONO HIP TRUSS
MONO HIP TRUSS
MONO HIP TRUSS
MONO HIP GIRDER
BASTARD HIP
BASTARD HIP
2'-0" CANTILEVER
1'-6" CANTILEVER
MONO TRUSSES @ 24" O.C.
3¼" X 11⅞" LSL RIM BEAM
5½" X 18" GLB LOW
3¼" X 11⅞" LSL
HUC66
DOUBLE JOIST
DOUBLE JOIST
MONO HIP GIRDER
MONO HIP TRUSS
MONO TRUSS
MONO TRUSS
MONO TRUSS
MONO GABLE END
P.T. 2 X 10 JOISTS @ 16" O.C.
LINE OF PATIO BELOW
TRIM ENDS OF DECK JOISTS TO MATCH ⅝ ROOF PITCH
3'-0 1/2"
8'-0"
8'-0"
6X12 DF#1
4X12
12"
6X8
6X12 DF#1
6X12 DF#1
5½X12 GLB


CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/
6 MIL. VAPOR BARRIER MINIMUM
P.T. 2 X 12 LEDGER
9½" I-JOISTS
13¾"x11⅞" LSL
19¾" X 11⅞" LSL
4 X 10
2 X 6 CRIPPLE WALL
4'-10"
11'-6"
11⅞" TJI 110 SERIES I-JSTS @ 16" O.C.
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
16
24
2'-6"
5'-0"
5'-0"
4'-3"
1'-0"
CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/
6 MIL. VAPOR BARRIER MINIMUM
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
3⅝" X 11⅞" LSL
6'-0"
6'-0"
2'-6"
2 X 6 CRIPPLE WALL
2'-6"
5'-6"
5'-6"
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
R-10 RIGID FOAM INSULATION OVER
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED FILL (TYP.)
14" X 7" SCREENED
FOUNDATION VENTS
(12) REQUIRED
3
CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/
6 MIL. VAPOR BARRIER MINIMUM
HUC48 W/ ¼"x1¾"
TITEN2 SCREWS
INTO CONCRETE
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY

MYERS ENGINEERING

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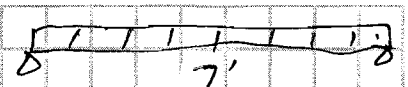
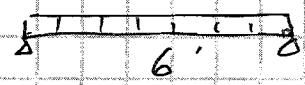
① $w_D = 15psf(40'/2) = 300plf$
$w_S = 25psf(40'/2) = 500plf$



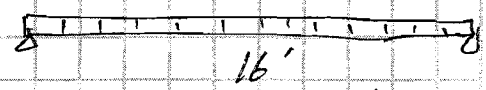
5 1/2 x 6" GLB
(4x10 @ 6' span)

② $w_D = 15psf(15'/2) = 112.5plf$
$w_S = 25psf(15'/2) = 187.5plf$



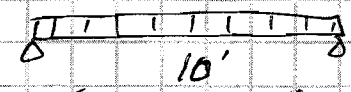
4x6

③ $w_D = 15psf$
$w_L = 40psf$



2x12 @ 16" o.c. Min.

④ $w_D = 10psf$
$w_L = 60psf$
$w_S = 25psf$

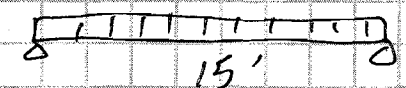


P.T. 2x10 @ 16" o.c.

⑤ $w_D = 15psf(12'/2) = 90plf$
$w_L = 60psf(10'/2) = 300plf$
$w_S = 25psf(12'/2) = 150plf$

5 1/2 x 12 GLB

⑥ $w_D = 15psf(7'/2) = 52.5plf$
$w_S = 25psf(7'/2) = 87.5plf$



6x8 DF#2

FOR Lot 4
JOB

DATE 7-28-20
BY MM

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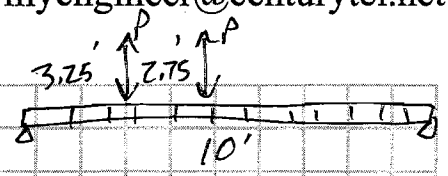
(7) $w_D = 15psf\left(15'/2 + 16'/2 + 10'/2\right) + 12psf(9') = 415.5plf$
$w_L = 40psf\left(16'/2\right) + 60psf\left(10'/2\right) = 620plf$
$w_S = 25psf\left(15'/2 + 10'/2\right) = 312.5plf$

$P = \pm 2850^{\#}WL \pm 2350^{\#}EL$ from wall $\Omega = 3.0$



6x12 DF#1

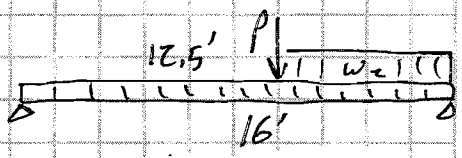
(8) $w_D = 15psf\left(27'/2\right) = 202.5plf$
$w_L = 40psf\left(27'/2\right) = 540plf$

5½x12 GLB

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

$P = 280^{\#}DL + 750^{\#}LL$

$w_{D2} = 15psf\left(6'/2\right) = 45plf$
$w_{L2} = 40psf\left(6'/2\right) = 120plf$



1¾x11⅞ LSL

(10) $w_{D1} = 15psf(2') + 12psf(9') = 138plf$
$w_{S1} = 25psf(2') = 50plf$

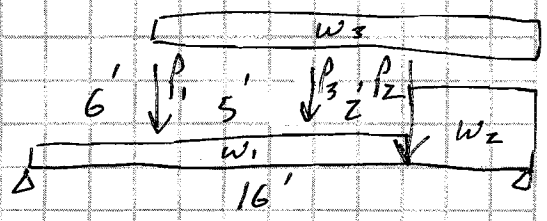
$w_{D2} = 15psf\left(40'/2\right) + 12psf(9') = 408plf$
$w_{S2} = 25psf\left(40'/2\right) = 500plf$

$w_{D3} = 15psf\left(12'/2\right) = 90plf$
$w_{S3} = 25psf\left(12'/2\right) = 150plf$

$P_1 = 410^{\#}DL + 595^{\#}SL$ from Hip Girder

$P_2 = 2105^{\#}DL + 3100^{\#}SL$ from Girder

$P_3 = \pm 3100^{\#}WL \pm 1650^{\#}EL$ from wall



5½x15" GLB

FOR Lot 4
JOB

DATE 7-28-20
BY MM

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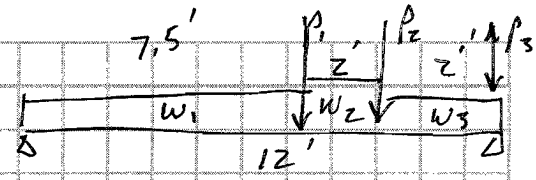
(11) $w_{D1} = 15psf(7'/2 + 12'/2) + 12psf(9') = 250.5plf$
$w_{L1} = 40psf(7'/2) = 140plf$
$w_{S1} = 25psf(12'/2) = 150plf$



$P_1 = 1050^{\#}DL + 1750^{\#}SL \pm 3100^{\#}WL \pm 1650^{\#}EL$ from (1) + wall 5½ x 13½ GLB

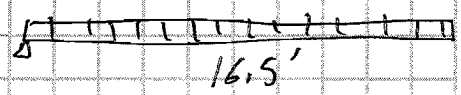
$w_{D2} = 15psf(40'/2 + 4'/2 + 12'/2) + 12psf(9') = 528plf$
$w_{L2} = 40psf(4'/2) = 80plf$
$w_{S2} = 25psf(40'/2 + 12'/2) = 650plf$

$P_2 = 1380^{\#}DL + 2025^{\#}SL$ from Girder

$w_{D3} = 15psf(2' + 4'/2 + 12'/2) + 12psf(9') = 258plf$
$w_{L3} = 80plf$
$w_{S3} = 25psf(2' + 12'/2) = 200plf$

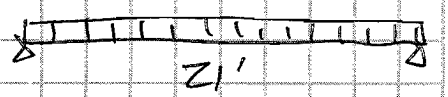
$P_3 = \pm 3100^{\#}WL \pm 1650^{\#}EL$ from wall

(12) $w_D = 15psf(12'/2 + 9'/2 + 1') + 12psf(9') = 280.5plf$
$w_L = 40psf(9'/2) = 180plf$
$w_S = 25psf(12'/2 + 1') = 175plf$



(3) 1¾ x 11⅞ LSL

(13) $w_D = 15psf(12'/2 + 8'/2 + 14'/2) + 12psf(9') = 363plf$
$w_L = 40psf(9'/2) = 180plf$
$w_S = 25psf(12'/2 + 14'/2) = 325plf$



5½ x 18 GLB

FOR Lot 4 DATE 7-28-20
JOB BY MM

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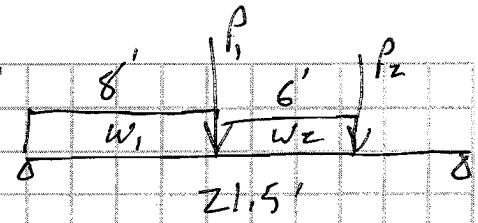
(14) $w_{D1} = 15psf(2' + 2' + 8'/2 + 12'/2) + 12psf(9') = 318plf$

$w_{L1} = 40psf(8'/2) = 160plf$

$w_{S1} = 25psf(2' + 2' + 12'/2) = 250plf$



$P_1 = 3812^{\#}DL + 1890^{\#}LL + 3413^{\#}SL$ from (13)

5 1/2 x 19 1/2 GLB

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

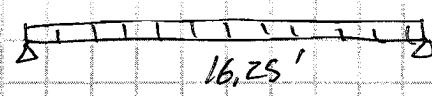
$w_{S2} = 25psf(12'/2) = 150plf$

$P_2 = 405^{\#}DL + 595^{\#}SL$ from hip girder

$w_{D3} = self$

(15) $w_D = 15psf(16'/2) = 120plf$

$w_S = 25psf(16'/2) = 200plf$

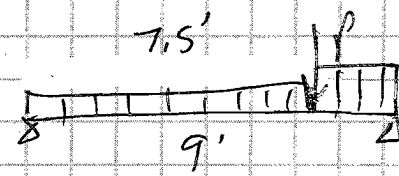


6x12 DF#1

(16) $w_{D1} = 15psf(2' + 1') + 12psf(9') = 153plf$

$w_{L1} = 40plf$

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



$P = 2315^{\#}DL + 1485^{\#}LL + 1445^{\#}SL$ from (12)

4x12

$w_{D2} = 15psf(19'/2) = 142.5plf$

$w_{S2} = 25psf(19'/2) = 237.5plf$

FOR Lot 4 DATE 7-29-20

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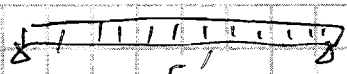
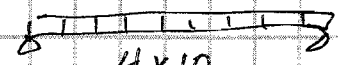
(17) $w_D = 15psf\left(\frac{30'}{2}\right) = 225plf$
$w_L = 40psf\left(\frac{30'}{2}\right) = 600plf$



5'

4x8 Min.

(18) $w_D = 15psf\left(\frac{20'}{2} + \frac{23'}{2}\right) = 322.5plf$
$w_L = 40psf\left(\frac{20'}{2} + \frac{23'}{2}\right) = 860plf$



4x10

5'6" Max Span

FOR Lot 4

JOB

DATE 7-29-20

BY MA

Mark Myers, PE
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Gig Harbor, WA 98335

Wood Beam

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DESCRIPTION: 1. Upper Floor Headers

CODE REFERENCES

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

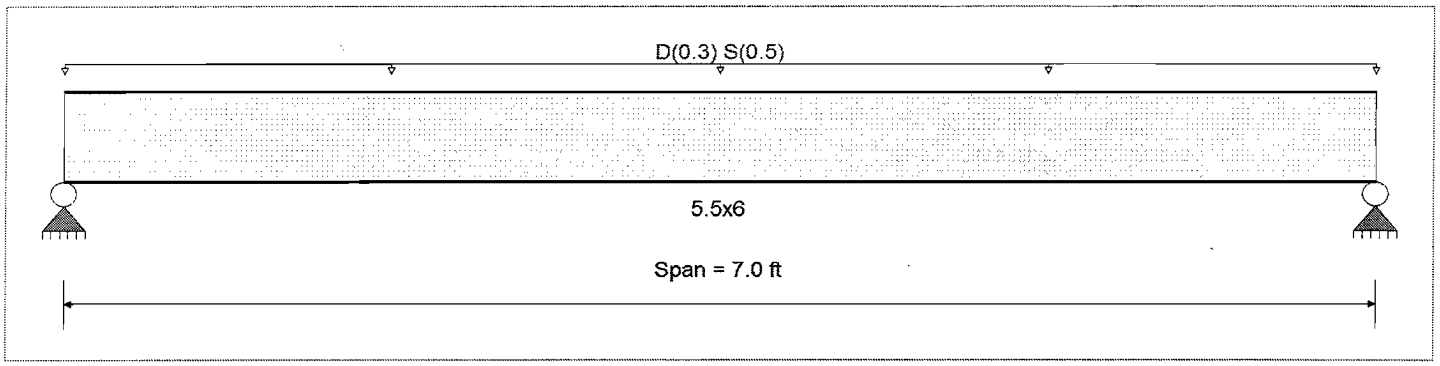
Material Properties

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



Applied Loads

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

Uniform Load : D = 0.30, S = 0.50 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.646 : 1**	Maximum Shear Stress Ratio	=	**0.360 : 1**
Section used for this span		**5.5x6**	Section used for this span		**5.5x6**
	=	1,781.82 psi		=	109.62 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.515 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.152 in	Ratio =	550 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.244 in	Ratio =	344 >=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	2.800	2.800
Overall MINimum	1.750	1.750
D Only	1.050	1.050
+D+L	1.050	1.050
+D+S	2.800	2.800
+D+0.750L	1.050	1.050
+D+0.750L+0.750S	2.363	2.363
+0.60D	0.630	0.630
S Only	1.750	1.750

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Wood Beam

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DESCRIPTION: 1a. Upper Floor Headers

CODE REFERENCES

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

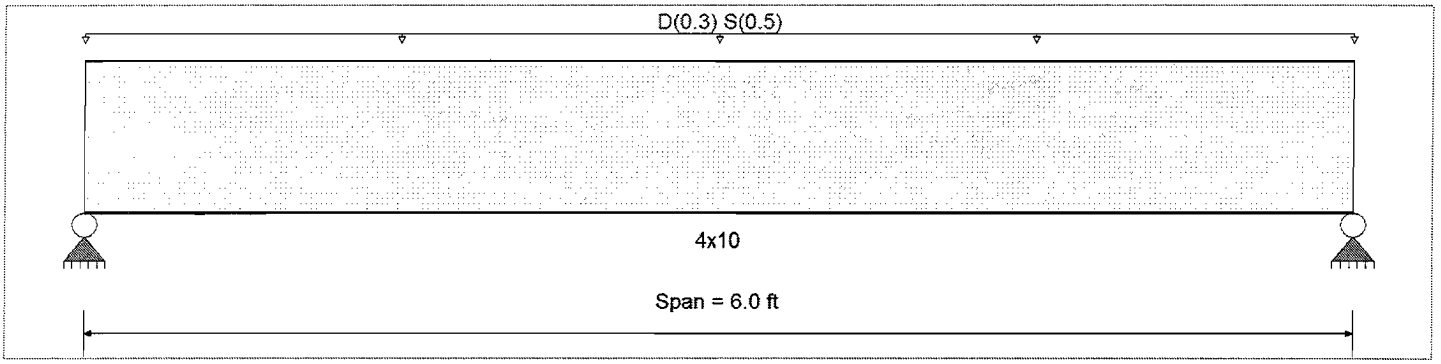
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Fb +	900 psi	*E : Modulus of Elasticity*	
Fb -	900 psi	Ebend- xx	1600 ksi
Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Fc - Perp	625 psi		
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 = 0.30, S = 0.50 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.697 : 1	Maximum Shear Stress Ratio	=	0.400 : 1
Section used for this span		4x10	Section used for this span		4x10
	=	865.53 psi		=	82.79 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	0.040 in	Ratio =	1813 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.064 in	Ratio =	1133 >=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	2.400	2.400
Overall MINimum	1.500	1.500
D Only	0.900	0.900
+D+L	0.900	0.900
+D+S	2.400	2.400
+D+0.750L	0.900	0.900
+D+0.750L+0.750S	2.025	2.025
+0.60D	0.540	0.540
S Only	1.500	1.500

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DESCRIPTION: 2. Upper Floor Headers

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

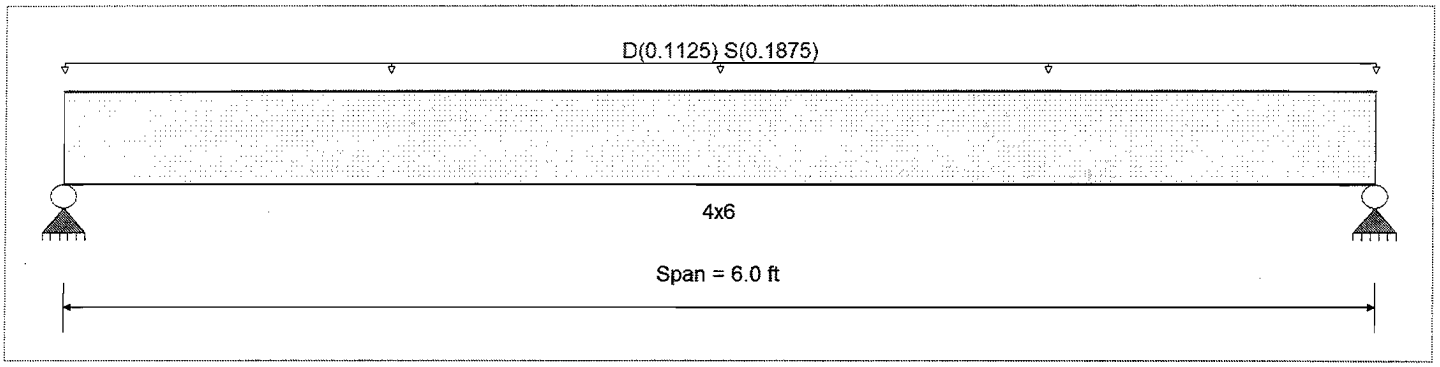
Wood Species : Douglas Fir - Larch
Wood Grade : No.2

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

Fb +	900.0 psi
Fb -	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	31.210 pcf



Applied Loads

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

Uniform Load : D = 0.1125, S = 0.1875 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.682 : 1**	Maximum Shear Stress Ratio	=	**0.289 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
	=	918.06 psi		=	59.89 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.071 in	Ratio =	1016 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.113 in	Ratio =	635 >=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	0.900	0.900
Overall MINimum	0.563	0.563
D Only	0.338	0.338
+D+L	0.338	0.338
+D+S	0.900	0.900
+D+0.750L	0.338	0.338
+D+0.750L+0.750S	0.759	0.759
+0.60D	0.203	0.203
S Only	0.563	0.563

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Wood Beam

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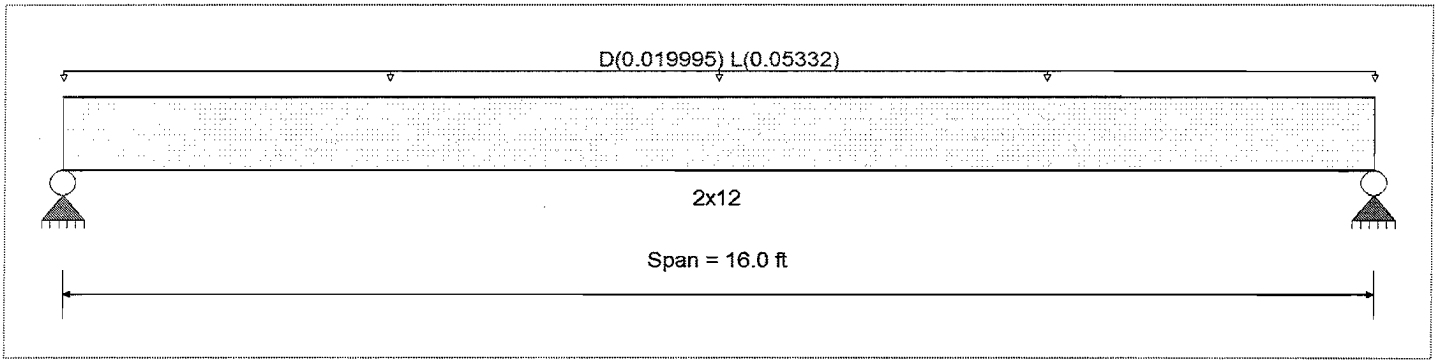
DESCRIPTION: 3. Floor Joists

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			Repetitive Member Stress Increase	



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.860 : 1**	Maximum Shear Stress Ratio	=	**0.256 : 1**
Section used for this span		**2x12**	Section used for this span		**2x12**
	=	889.77 psi		=	46.05 psi
	=	1,035.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.278 in	Ratio =	691 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.382 in	Ratio =	502 >=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	0.587	0.587
Overall MINimum	0.427	0.427
D Only	0.160	0.160
+D+L	0.587	0.587
+D+S	0.160	0.160
+D+0.750L	0.480	0.480
+D+0.750L+0.750S	0.480	0.480
+0.60D	0.096	0.096
L Only	0.427	0.427
S Only		

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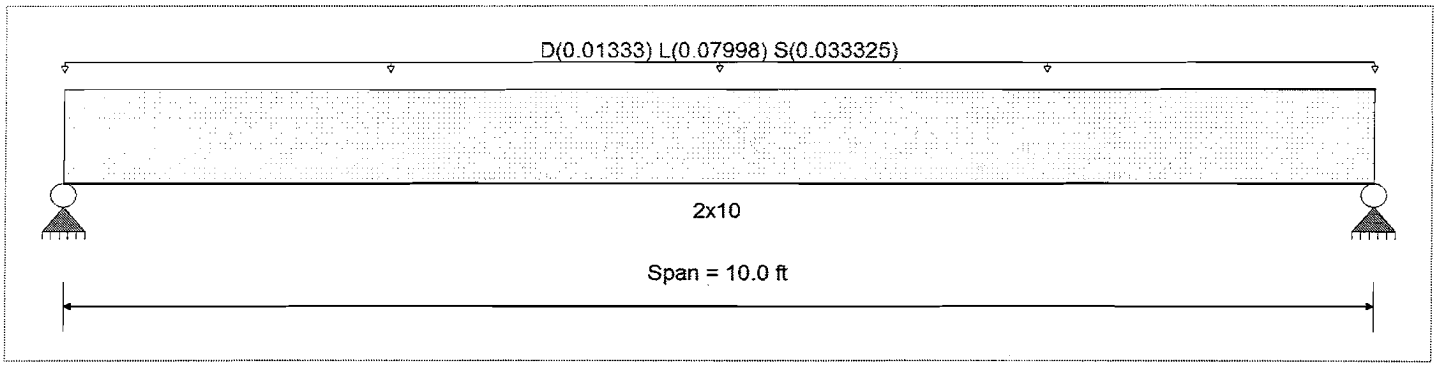
DESCRIPTION: 4. Deck Joists

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



Applied Loads

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

Uniform Load : D = 0.010, L = 0.060, S = 0.0250 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.761: 1**	Maximum Shear Stress Ratio	=	**0.356 : 1**
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	654.33 psi		=	42.71 psi
	=	860.20 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.148 in	Ratio =	810 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.182 in	Ratio =	658 >=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	0.492	0.492
Overall MINimum	0.167	0.167
D Only	0.067	0.067
+D+L	0.467	0.467
+D+S	0.233	0.233
+D+0.750L	0.367	0.367
+D+0.750L+0.750S	0.492	0.492
+0.60D	0.040	0.040
L Only	0.400	0.400
S Only	0.167	0.167

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Wood Beam

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MYERS ENGINEERING

DESCRIPTION: 5. Deck Beam

CODE REFERENCES

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

Material Properties

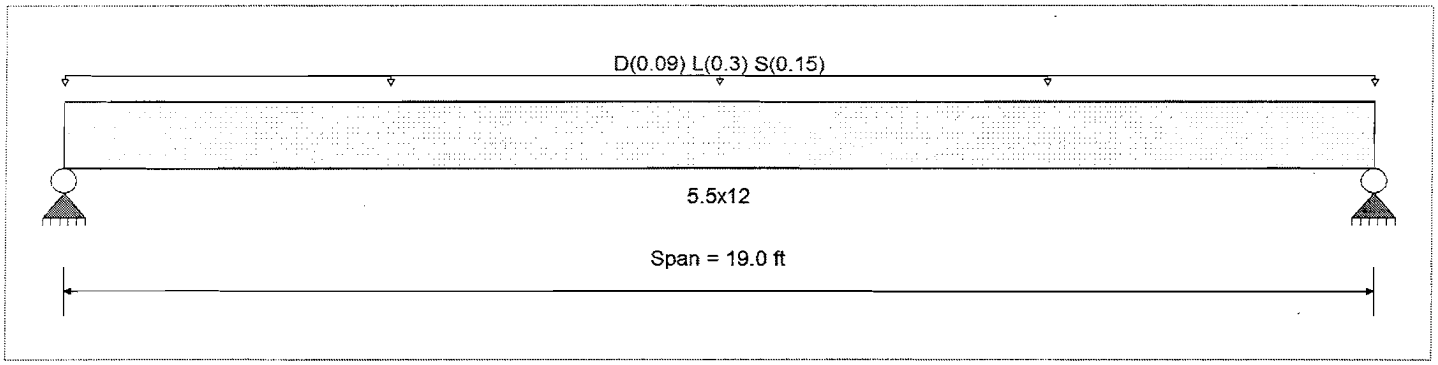
Analysis Method : Allowable Stress Design
Load Combination IBC 2018

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.

Uniform Load : D = 0.090, L = 0.30, S = 0.150 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.667 : 1**	Maximum Shear Stress Ratio	=	**0.285 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,599.89 psi		=	75.60 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.500 ft	Location of maximum on span	=	18.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.621 in	Ratio =	367 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.884 in	Ratio =	257 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.061	4.061
Overall MINimum	1.425	1.425
D Only	0.855	0.855
+D+L	3.705	3.705
+D+S	2.280	2.280
+D+0.750L	2.993	2.993
+D+0.750L+0.750S	4.061	4.061
+0.60D	0.513	0.513
L Only	2.850	2.850
S Only	1.425	1.425

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Wood Beam

DESCRIPTION: 6. Cov'd Porch Roof Beam

CODE REFERENCES

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

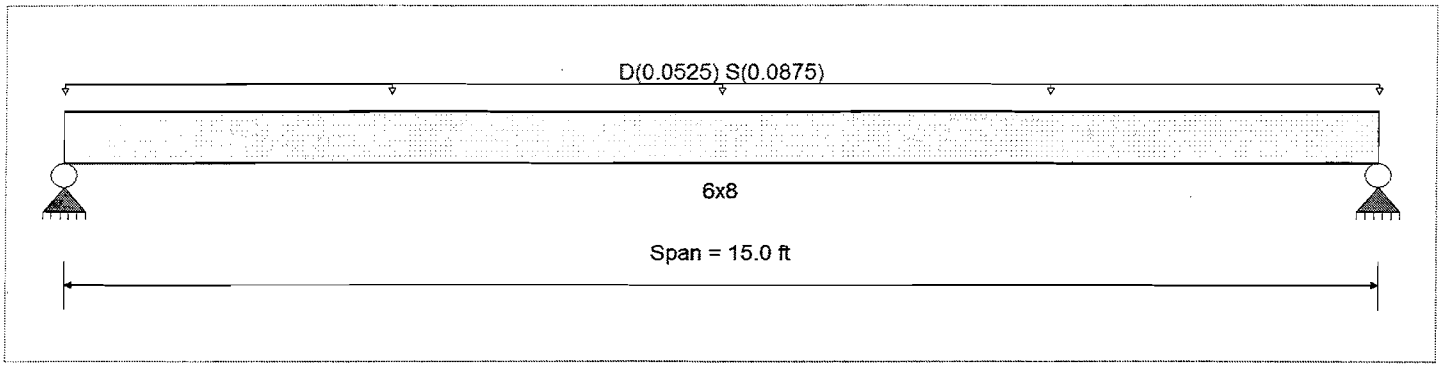
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

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

Fb +	875 psi	*E : Modulus of Elasticity*	
Fb -	875 psi	Ebend- xx	1300 ksi
Fc - Prll	600 psi	Eminbend - xx	470 ksi
Fc - Perp	625 psi		
Fv	170 psi		
Ft	425 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.05250, S = 0.08750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.911: 1	Maximum Shear Stress Ratio	=	0.180 : 1
Section used for this span		6x8	Section used for this span		6x8
	=	916.36 psi		=	35.12 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.500 ft	Location of maximum on span	=	14.398 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.399 in	Ratio =	451 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.638 in	Ratio =	282 >=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	1.050	1.050
Overall MINimum	0.656	0.656
D Only	0.394	0.394
+D+L	0.394	0.394
+D+S	1.050	1.050
+D+0.750L	0.394	0.394
+D+0.750L+0.750S	0.886	0.886
+0.60D	0.236	0.236
S Only	0.656	0.656

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Wood Beam

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DESCRIPTION: 7. Header at accordion door

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

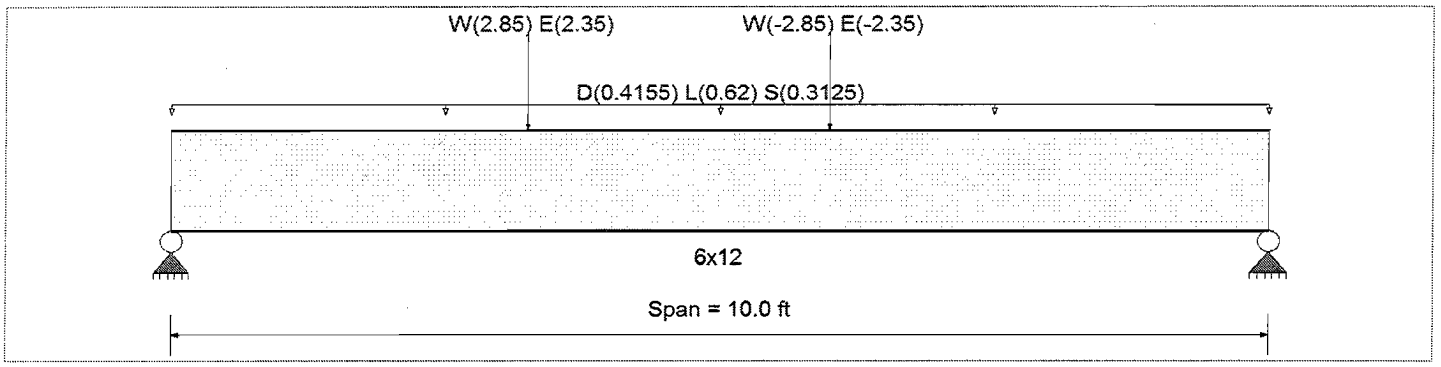
Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Fb +	1350 psi
Fb -	1350 psi
Fc - Prll	925 psi
Fc - Perp	625 psi
Fv	170 psi
Ft	675 psi

E : Modulus of Elasticity

Ebend- xx	1600 ksi
Eminbend - xx	580 ksi
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 = 0.4155, L = 0.620, S = 0.3125 , Tributary Width = 1.0 ft
Point Load : W = 2.850, E = 2.350 k @ 3.250 ft
Point Load : W = -2.850, E = -2.350 k @ 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.949 : 1	Maximum Shear Stress Ratio	=	0.585 : 1
Section used for this span		6x12	Section used for this span		6x12
	=	1,281.25 psi		=	99.48 psi
	=	1,350.00 psi		=	170.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.126 in	Ratio =	953 >=480
Max Upward Transient Deflection	-0.015 in	Ratio =	7813 >=480
Max Downward Total Deflection	0.231 in	Ratio =	519 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.927	5.927
Overall MINimum	-0.784	0.646
D Only	2.078	2.078
+D+L	5.178	5.178
+D+S	3.640	3.640
+D+0.750L	4.403	4.403
+D+0.750L+0.750S	5.574	5.574
+D+0.60W	2.548	1.607
+D-0.60W	1.607	2.548
+D+0.70E	2.530	1.625
+D-0.70E	1.625	2.530
+D+0.750L+0.450W	4.755	4.050
+D+0.750L-0.450W	4.050	4.755

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DESCRIPTION: 7. Header at accordion door

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W	5.927	5.222
+D+0.750L+0.750S-0.450W	5.222	5.927
+D+0.750L+0.750S+0.5250E	5.914	5.235
+D+0.750L+0.750S-0.5250E	5.235	5.914
+0.60D+0.60W	1.717	0.776
+0.60D-0.60W	0.776	1.717
+0.60D+0.70E	1.699	0.794
+0.60D-0.70E	0.794	1.699
L Only	3.100	3.100
S Only	1.563	1.563
W Only	0.784	-0.784
-W	-0.784	0.784
E Only	0.646	-0.646
E Only * -1.0	-0.646	0.646

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Wood Beam

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DESCRIPTION: 8. Beam at Foyer/Living 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

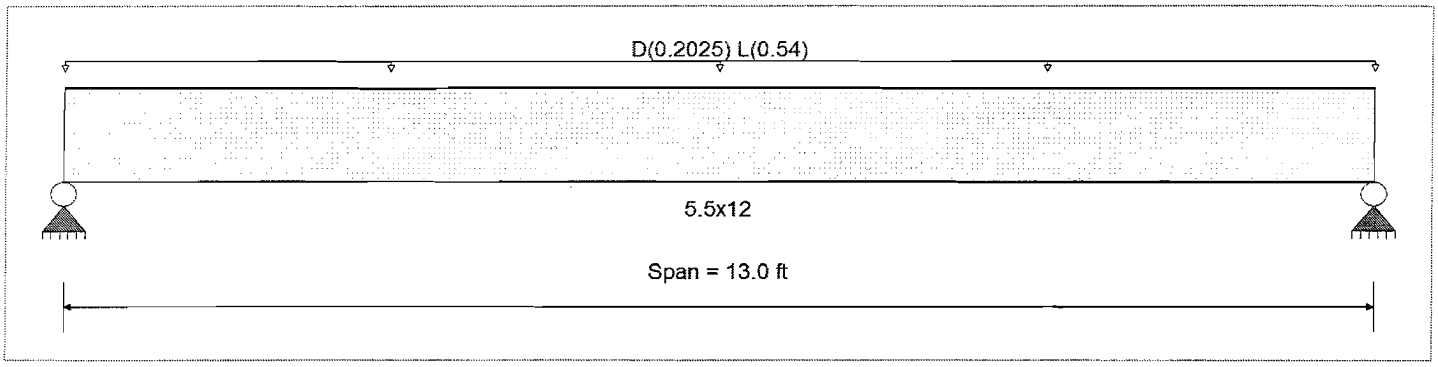
Wood Species : DF/DF
Wood Grade : 24F - V4

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 = 0.2025, L = 0.540 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.594 : 1**	Maximum Shear Stress Ratio	=	**0.350 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,425.94 psi		=	92.87 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.500 ft	Location of maximum on span	=	12.004 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.245 in	Ratio =	637 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.337 in	Ratio =	463 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.826	4.826
Overall MINimum	3.510	3.510
D Only	1.316	1.316
+D+L	4.826	4.826
+D+S	1.316	1.316
+D+0.750L	3.949	3.949
+D+0.750L+0.750S	3.949	3.949
+0.60D	0.790	0.790
L Only	3.510	3.510
S Only		

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Wood Beam

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DESCRIPTION: 9. Floor beam at shower

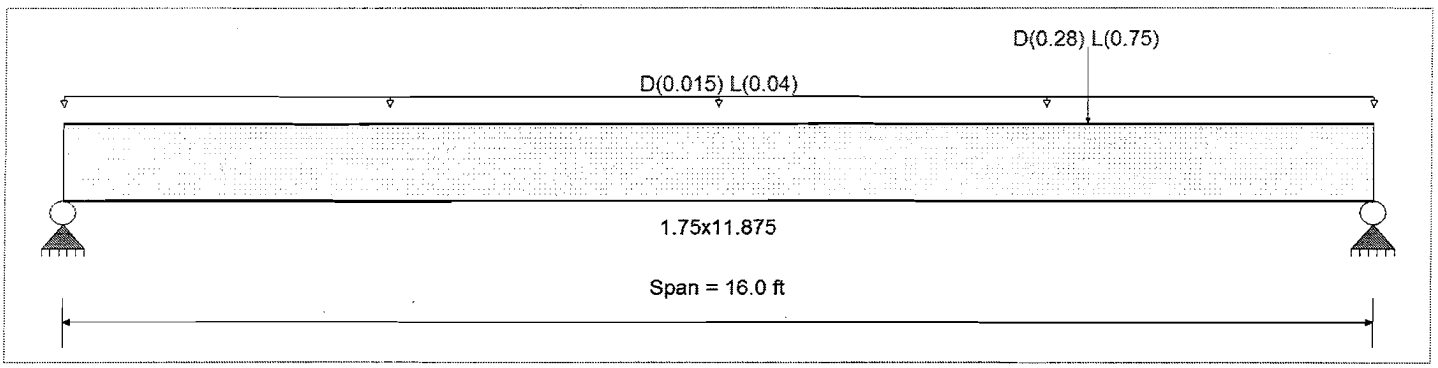
CODE REFERENCES

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 = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.280, L = 0.750 k @ 12.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.505 : 1**	Maximum Shear Stress Ratio	=	**0.278 : 1**
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,174.05 psi		=	86.13 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	12.088 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.339 in	Ratio =	566 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.466 in	Ratio =	412 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.665	1.245
Overall MINimum	0.484	0.906
D Only	0.181	0.339
+D+L	0.665	1.245
+D+S	0.181	0.339
+D+0.750L	0.544	1.018
+D+0.750L+0.750S	0.544	1.018
+0.60D	0.109	0.203
L Only	0.484	0.906
S Only		

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Wood Beam

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DESCRIPTION: 10. Rim beam over Kitchen

CODE REFERENCES

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

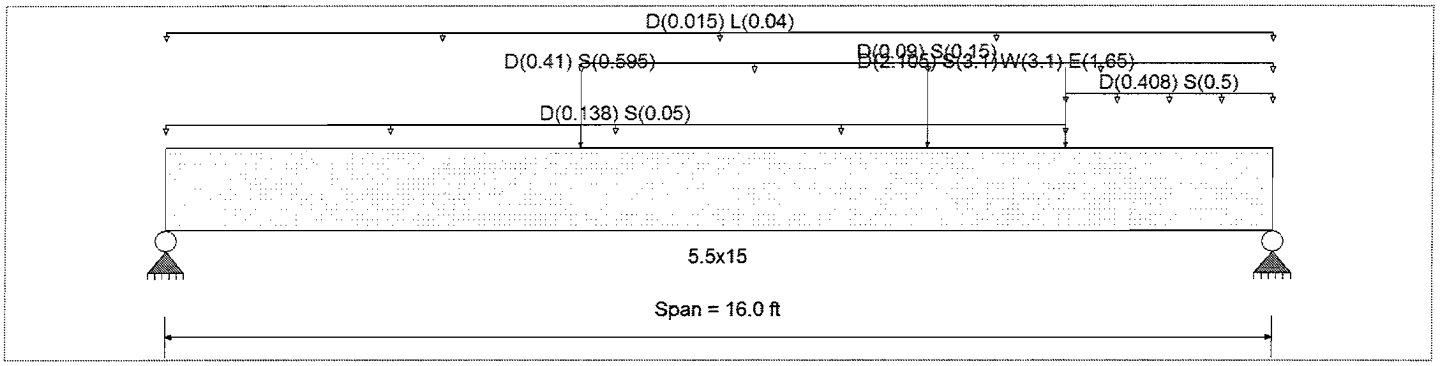
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

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

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

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 = 0.1380, S = 0.050 k/ft, Extent = 0.0 –>> 13.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.410, S = 0.5950 k @ 6.0 ft
- Uniform Load : D = 0.4080, S = 0.50 k/ft, Extent = 13.0 –>> 16.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.090, S = 0.150 k/ft, Extent = 6.0 –>> 16.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
- Point Load : D = 2.105, S = 3.10 k @ 11.0 ft
- Point Load : W = 3.10, E = 1.650 k @ 13.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.694 : 1**	Maximum Shear Stress Ratio	=	**0.463 : 1**
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	1,909.98 psi		=	141.10 psi
	=	2,753.98 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.978 ft	Location of maximum on span	=	14.774 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.267 in	Ratio =	720 >=480
Max Upward Transient Deflection	-0.090 in	Ratio =	2127 >=480
Max Downward Total Deflection	0.505 in	Ratio =	380 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.831	9.308
Overall MINimum	-0.581	-2.519
D Only	2.495	4.178
+D+L	2.815	4.498
+D+S	4.831	9.187
+D+0.750L	2.735	4.418
+D+0.750L+0.750S	4.487	8.175
+D+0.60W	2.844	5.689

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Wood Beam

DESCRIPTION: 10. Rim beam over Kitchen

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D-0.60W	2.147	2.667
+D+0.70E	2.712	5.116
+D-0.70E	2.279	3.239
+D+0.750L+0.450W	2.997	5.551
+D+0.750L-0.450W	2.474	3.284
+D+0.750L+0.750S+0.450W	4.749	9.308
+D+0.750L+0.750S-0.450W	4.226	7.041
+D+0.750L+0.750S+0.5250E	4.650	8.878
+D+0.750L+0.750S-0.5250E	4.325	7.471
+0.60D+0.60W	1.846	4.018
+0.60D-0.60W	1.148	0.995
+0.60D+0.70E	1.714	3.445
+0.60D-0.70E	1.281	1.568
L Only	0.320	0.320
S Only	2.336	5.009
W Only	0.581	2.519
-W	-0.581	-2.519
E Only	0.309	1.341
E Only * -1.0	-0.309	-1.341

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Wood Beam

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DESCRIPTION: 11. Beam over Pantry/Laundry

CODE REFERENCES

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

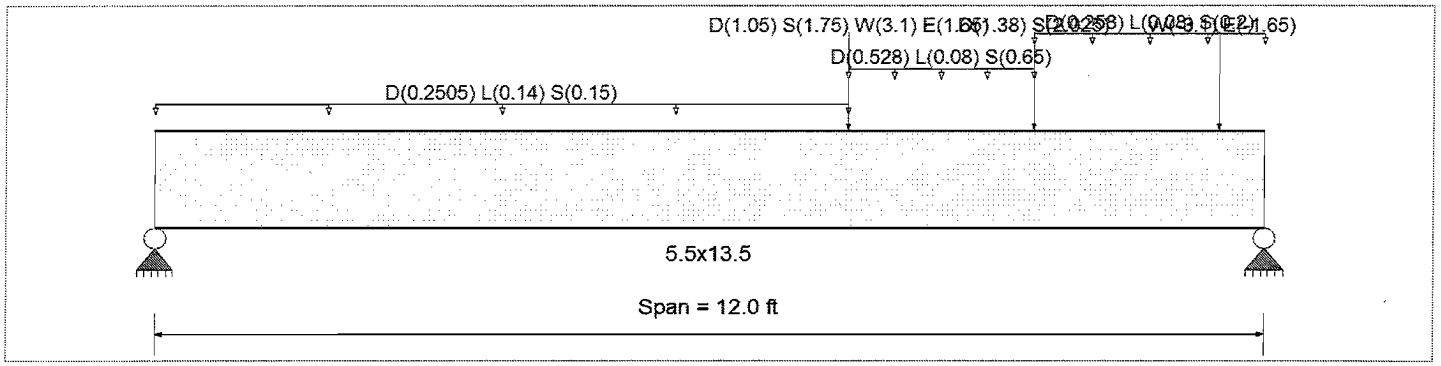
Material Properties

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



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.2505, L = 0.140, S = 0.150 k/ft, Extent = 0.0 –>> 7.50 ft, Tributary Width = 1.0 ft
- Point Load : D = 1.050, S = 1.750, W = 3.10, E = 1.650 k @ 7.50 ft
- Uniform Load : D = 0.5280, L = 0.080, S = 0.650 k/ft, Extent = 7.50 –>> 9.50 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.2580, L = 0.080, S = 0.20 k/ft, Extent = 9.50 –>> 12.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 1.380, S = 2.025 k @ 9.50 ft
- Point Load : W = -3.10, E = -1.650 k @ 11.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.610 : 1**	Maximum Shear Stress Ratio	=	**0.502 : 1**
Section used for this span		**5.5x13.5**	Section used for this span		**5.5x13.5**
	=	1,684.38 psi		=	153.08 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.489 ft	Location of maximum on span	=	10.905 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.146 in	Ratio =	983 >=480
Max Upward Transient Deflection	-0.076 in	Ratio =	1901 >=480
Max Downward Total Deflection	0.291 in	Ratio =	494 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.117	8.079
Overall MINimum	-1.033	0.550
D Only	2.348	3.662
+D+L	3.137	4.282
+D+S	4.631	8.079
+D+0.750L	2.940	4.127
+D+0.750L+0.750S	4.652	7.440
+D+0.60W	2.968	3.042
+D-0.60W	1.728	4.282

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DESCRIPTION: 11. Beam over Pantry/Laundry

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.70E	2.733	3.277
+D-0.70E	1.963	4.047
+D+0.750L+0.450W	3.405	3.662
+D+0.750L-0.450W	2.475	4.592
+D+0.750L+0.750S+0.450W	5.117	6.975
+D+0.750L+0.750S-0.450W	4.187	7.905
+D+0.750L+0.750S+0.5250E	4.941	7.151
+D+0.750L+0.750S-0.5250E	4.363	7.729
+0.60D+0.60W	2.029	1.577
+0.60D-0.60W	0.789	2.817
+0.60D+0.70E	1.794	1.812
+0.60D-0.70E	1.024	2.582
L Only	0.789	0.621
S Only	2.283	4.417
W Only	1.033	-1.033
-W	-1.033	1.033
E Only	0.550	-0.550
E Only * -1.0	-0.550	0.550

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Wood Beam

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DESCRIPTION: 12. Rim Beam over Master

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

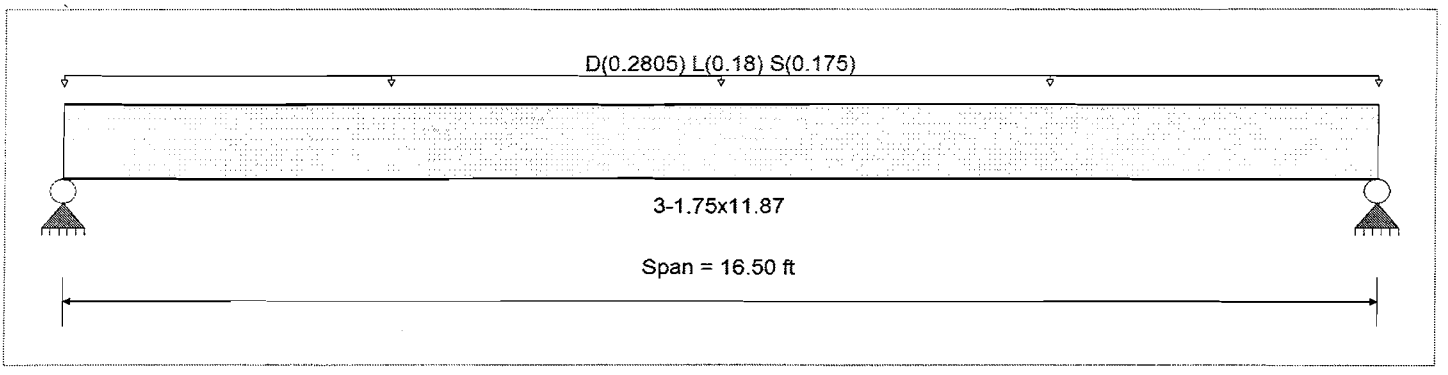
Wood Species : Trus Joist
Wood Grade : TimberStrand LSL 1.55E

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

Fb +	2325 psi
Fb -	2325 psi
Fc - Prll	2170 psi
Fc - Perp	900 psi
Fv	310 psi
Ft	1070 psi

E : Modulus of Elasticity

Ebend- xx	1550 ksi
Eminbend - xx	787.815 ksi
Density	45.01 pcf



Applied Loads

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

Uniform Load : D = 0.2805, L = 0.180, S = 0.1750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.677 : 1**	Maximum Shear Stress Ratio	=	**0.269 : 1**
Section used for this span		**3-1.75x11.87**	Section used for this span		**3-1.75x11.87**
	=	1,809.56 psi		=	95.85 psi
	=	2,673.75 psi		=	356.50 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.266 in	Ratio =	744 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.808 in	Ratio =	245 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.511	4.511
Overall MINimum	1.444	1.444
D Only	2.314	2.314
+D+L	3.799	3.799
+D+S	3.758	3.758
+D+0.750L	3.428	3.428
+D+0.750L+0.750S	4.511	4.511
+0.60D	1.388	1.388
L Only	1.485	1.485
S Only	1.444	1.444

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Wood Beam

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DESCRIPTION: 13. Beam over Garage at Grid 3

CODE REFERENCES

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

Material Properties

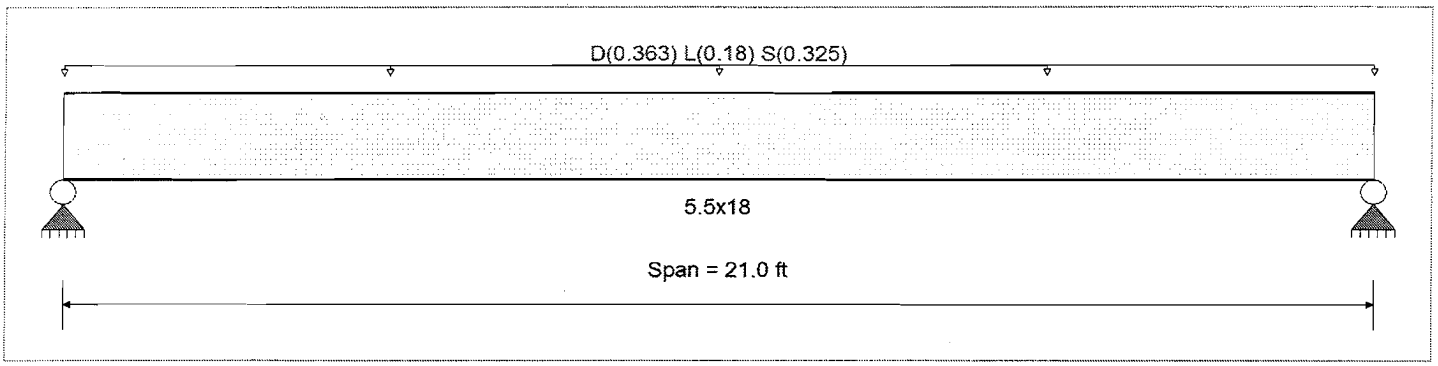
Analysis Method : Allowable Stress Design
Load Combination IBC 2018

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.

Uniform Load : D = 0.3630, L = 0.180, S = 0.3250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.628 : 1**	Maximum Shear Stress Ratio	=	**0.334 : 1**
Section used for this span		**5.5x18**	Section used for this span		**5.5x18**
	=	1,652.08 psi		=	101.64 psi
	=	2,631.68 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	10.500 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	0.297 in	Ratio =	847 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.679 in	Ratio =	371 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	7.788	7.788
Overall MINimum	3.413	3.413
D Only	3.812	3.812
+D+L	5.702	5.702
+D+S	7.224	7.224
+D+0.750L	5.229	5.229
+D+0.750L+0.750S	7.788	7.788
+0.60D	2.287	2.287
L Only	1.890	1.890
S Only	3.413	3.413

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Wood Beam

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DESCRIPTION: 14. Beam over Garage at Grid F

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

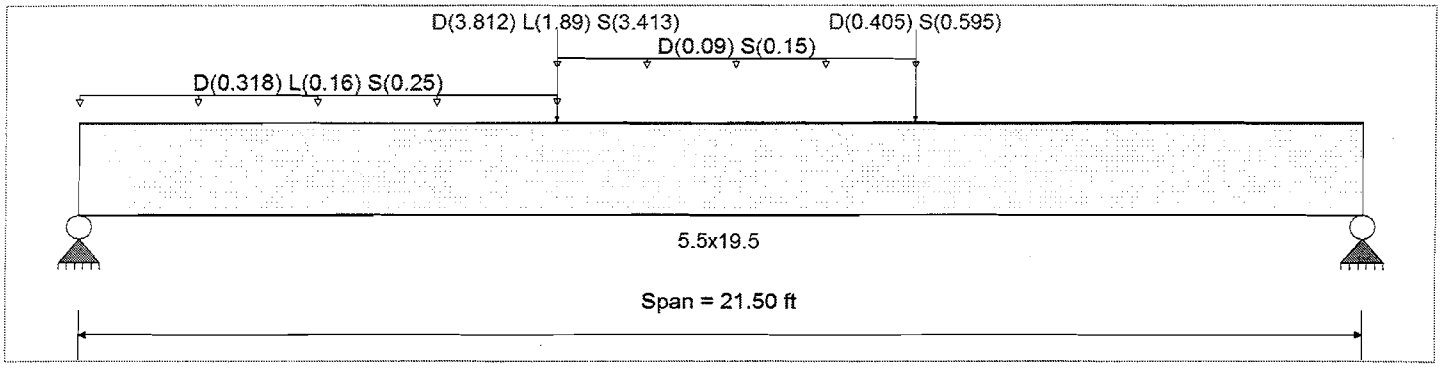
Wood Species : DF/DF
Wood Grade : 24F - V4

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.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.3180, L = 0.160, S = 0.250 k/ft, Extent = 0.0 -->> 8.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.090, S = 0.150 k/ft, Extent = 8.0 -->> 14.0 ft, Tributary Width = 1.0 ft
Point Load : D = 3.812, L = 1.890, S = 3.413 k @ 8.0 ft
Point Load : D = 0.4050, S = 0.5950 k @ 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.794 : 1**	Maximum Shear Stress Ratio	=	**0.417 : 1**
Section used for this span		**5.5x19.5**	Section used for this span		**5.5x19.5**
	=	2,067.79 psi		=	127.08 psi
	=	2,604.56 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.004 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.325 in	Ratio =	793 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.701 in	Ratio =	368 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	10.104	5.255
Overall MINimum	4.418	2.490
D Only	5.119	2.682
+D+L	7.348	3.623
+D+S	9.537	5.172
+D+0.750L	6.791	3.388
+D+0.750L+0.750S	10.104	5.255
+0.60D	3.071	1.609
L Only	2.229	0.941
S Only	4.418	2.490

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Wood Beam

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DESCRIPTION: 15. Garage Door Header

CODE REFERENCES

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

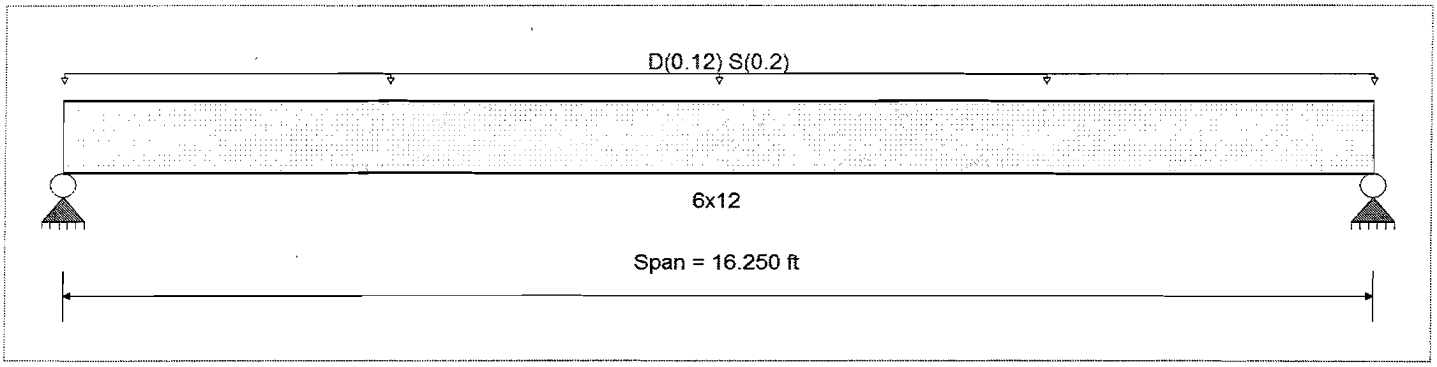
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

Wood Species : Douglas Fir - Larch
Wood Grade : No.1

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

Fb +	1350 psi	*E : Modulus of Elasticity*	
Fb -	1350 psi	Ebend- xx	1600 ksi
Fc - Prll	925 psi	Eminbend - xx	580 ksi
Fc - Perp	625 psi		
Fv	170 psi		
Ft	675 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.120, S = 0.20 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.673 : 1**	Maximum Shear Stress Ratio	=	**0.279 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	1,045.54 psi		=	54.46 psi
	=	1,552.50 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.283 in	Ratio =	689 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.453 in	Ratio =	430 >=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	2.600	2.600
Overall MINimum	1.625	1.625
D Only	0.975	0.975
+D+L	0.975	0.975
+D+S	2.600	2.600
+D+0.750L	0.975	0.975
+D+0.750L+0.750S	2.194	2.194
+0.60D	0.585	0.585
S Only	1.625	1.625

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Wood Beam

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DESCRIPTION: 16. Header at Master Bedroom

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

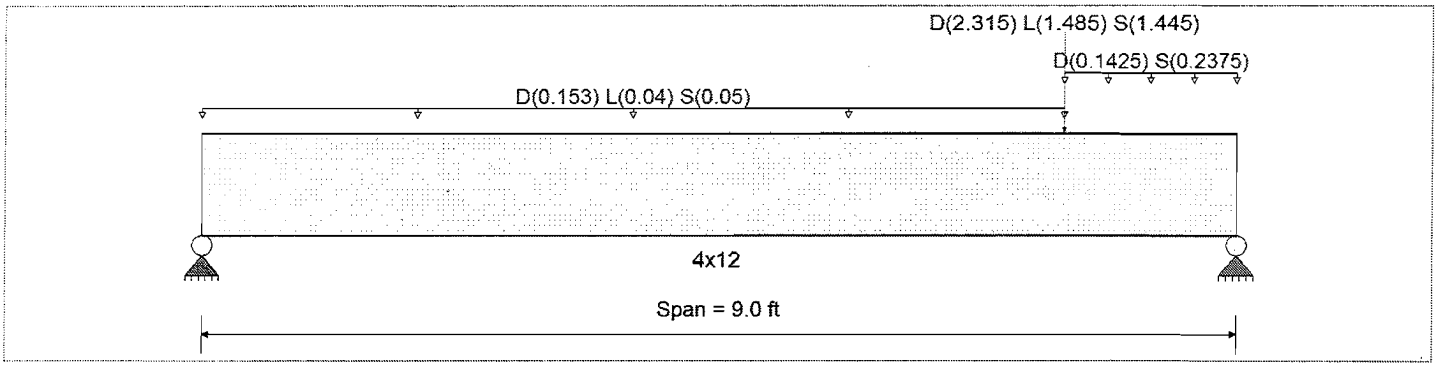
Wood Species : DouglasFir-Larch
Wood Grade : No.2

Fb +	900.0 psi
Fb -	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
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.

Load for Span Number 1
Uniform Load : D = 0.1530, L = 0.040, S = 0.050 k/ft, Extent = 0.0 -->> 7.50 ft, Tributary Width = 1.0 ft
Point Load : D = 2.315, L = 1.485, S = 1.445 k @ 7.50 ft
Uniform Load : D = 0.1425, S = 0.2375 k/ft, Extent = 7.50 -->> 9.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.996 : 1	Maximum Shear Stress Ratio	=	0.846 : 1
Section used for this span		4x12	Section used for this span		4x12
	=	1,133.50 psi		=	175.07 psi
	=	1,138.50 psi		=	207.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	7.489 ft	Location of maximum on span	=	8.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.042 in	Ratio =	2571 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.138 in	Ratio =	781 >=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	1.757	4.890
Overall MINimum	0.489	1.687
D Only	1.073	2.603
+D+L	1.496	3.966
+D+S	1.562	4.290
+D+0.750L	1.390	3.625
+D+0.750L+0.750S	1.757	4.890
+0.60D	0.644	1.562
L Only	0.423	1.363
S Only	0.489	1.687

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Wood Beam

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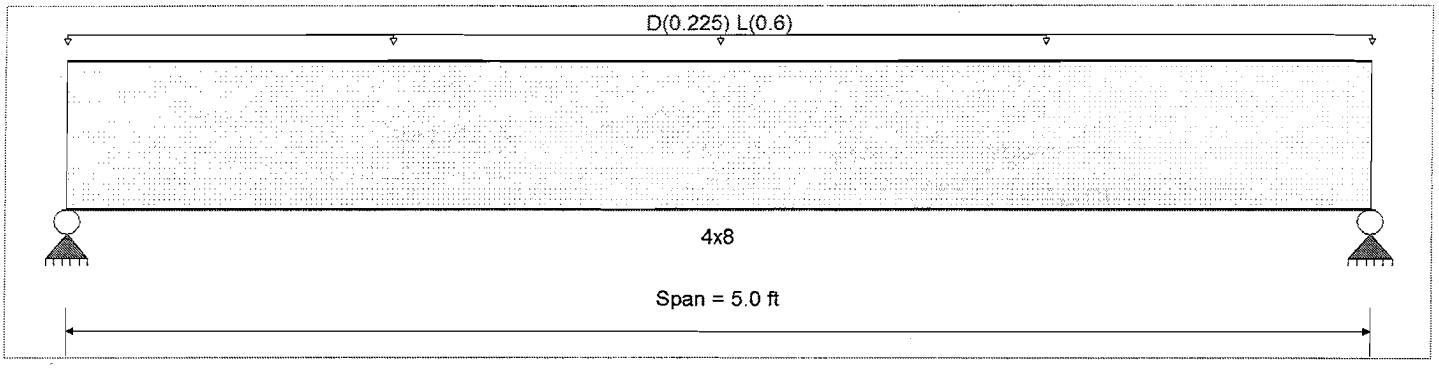
DESCRIPTION: 17. Crawl beam at Kitchen/Pantry

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 = 0.2250, L = 0.60 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.862 1**	Maximum Shear Stress Ratio	=	**0.514 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,009.00 psi		=	92.55 psi
	=	1,170.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.500 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	0.048 in	Ratio =	1257 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.066 in	Ratio =	914 >=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	2.063	2.063
Overall MINimum	1.500	1.500
D Only	0.563	0.563
+D+L	2.063	2.063
+D+S	0.563	0.563
+D+0.750L	1.688	1.688
+D+0.750L+0.750S	1.688	1.688
+0.60D	0.338	0.338
L Only	1.500	1.500
S Only		

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Wood Beam

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DESCRIPTION: 18. Crawl beam

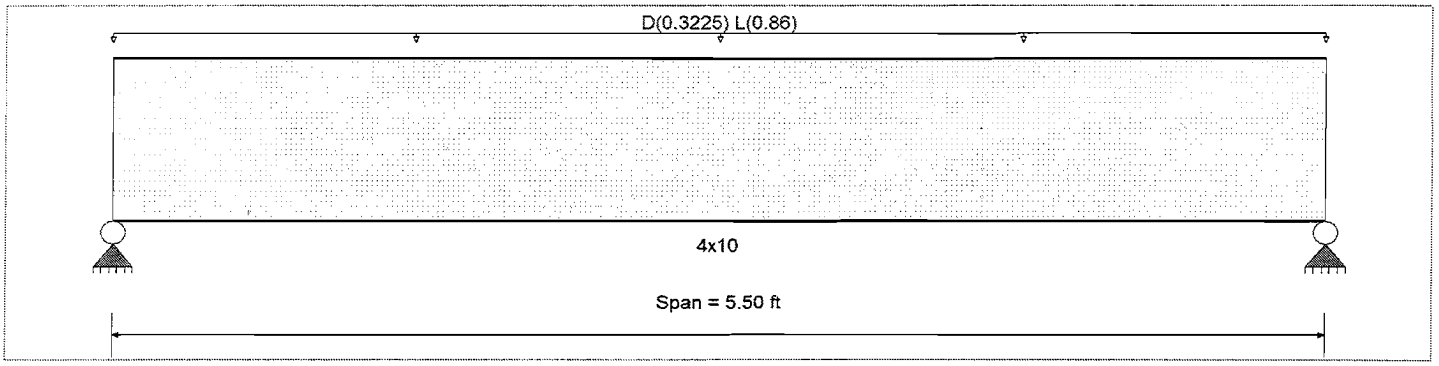
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 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 = 0.3225, L = 0.860 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.995 : 1**	Maximum Shear Stress Ratio	=	**0.605 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,075.02 psi		=	108.88 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.750 ft	Location of maximum on span	=	4.737 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.048 in	Ratio =	1368 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.066 in	Ratio =	995 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.252	3.252
Overall MINimum	2.365	2.365
D Only	0.887	0.887
+D+L	3.252	3.252
+D+S	0.887	0.887
+D+0.750L	2.661	2.661
+D+0.750L+0.750S	2.661	2.661
+0.60D	0.532	0.532
L Only	2.365	2.365
S Only		

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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)

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

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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)