MYERS ENGINEERING

LATERAL ANALYSIS & GRAVITY CALCULATIONS

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Date: 2020.11.24 17:45:03 -08'00'

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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: RKK – Lot 2
3402 72nd Place Southeast
Mercer Island, WA

November 24, 2020

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

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

Myers Engineering, LLC
3206 50th Street Ct NW, Ste 210-B
Gig Harbor, WA 98335
Phone: 253-858-3248
Email: myengineer@centurytel.net

PROJECT : **3402 72nd Place SE**

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

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

WOODS : WOOD TYPE:

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

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

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

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

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

TRUSSES:

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

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

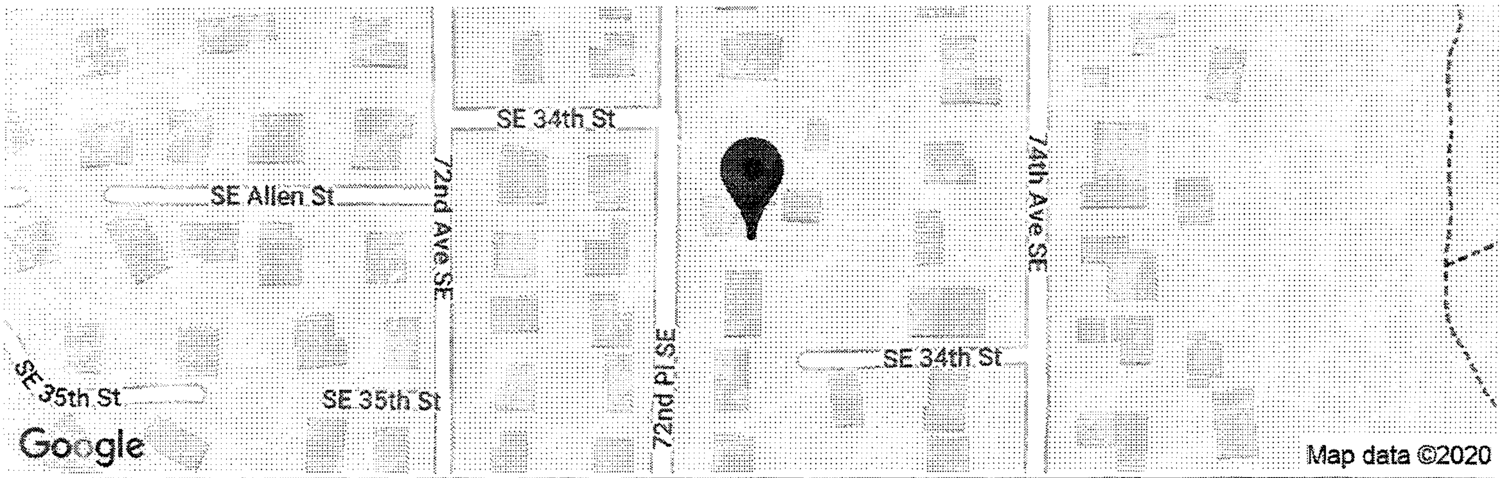
ENGINEERED I-JOISTS

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

OSHPD

340X 72nd Place SE

Latitude, Longitude: 47.57962474, -122.24174847



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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	1.455	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.805	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	1.164	Factored deterministic acceleration value. (1.0 second)
PGAd	1.077	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.959	Mapped value of the risk coefficient at short periods
C_{R1}	0.934	Mapped value of the risk coefficient at a period of 1 s

LATERAL ANALYSIS :
BASED ON 2015 INTERNATIONAL BUILDING CODE (IBC)

Lateral Forces will be distributed along lines of Force/Resistance. Lines of Force/Resistance will be investigated for both wind and seismic lateral loads. Roof and Floor diaphragms are considered flexible.

Risk Category II per IBC 1604.5 & Soils Site Class D (Assumed)

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

Seismic Design Data:

$I_e := 1.0$ (ASCE 7-10 Table 1.5-2)

$R := 6.5$ $\Omega_0 := 3.0$ $C_d := 4$ Light-frame (wood) walls sheathed w/ wood structural panels rated for shear resistance (ASCE 7-10 Table 12.2-1)

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

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: $S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{8}{12}\right)\right)} = 1.2$

Plan Area for Each Level:

$A_1 := 1970\text{ft}^2 \cdot S_a$ (Upper Roof) $A_{2a} := 1776\text{ft}^2$ (Framed Floor) $A_{2b} := 338\text{ft}^2$ (Deck)

Plan Perimeter for Each Level:

$P_1 := 2(40\text{ft}) + 2(55\text{ft})$ (Upper Floor) $P_2 := 2(40\text{ft}) + 2(55\text{ft})$ (Main Floor)

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

Weight of Structure at Each Level:

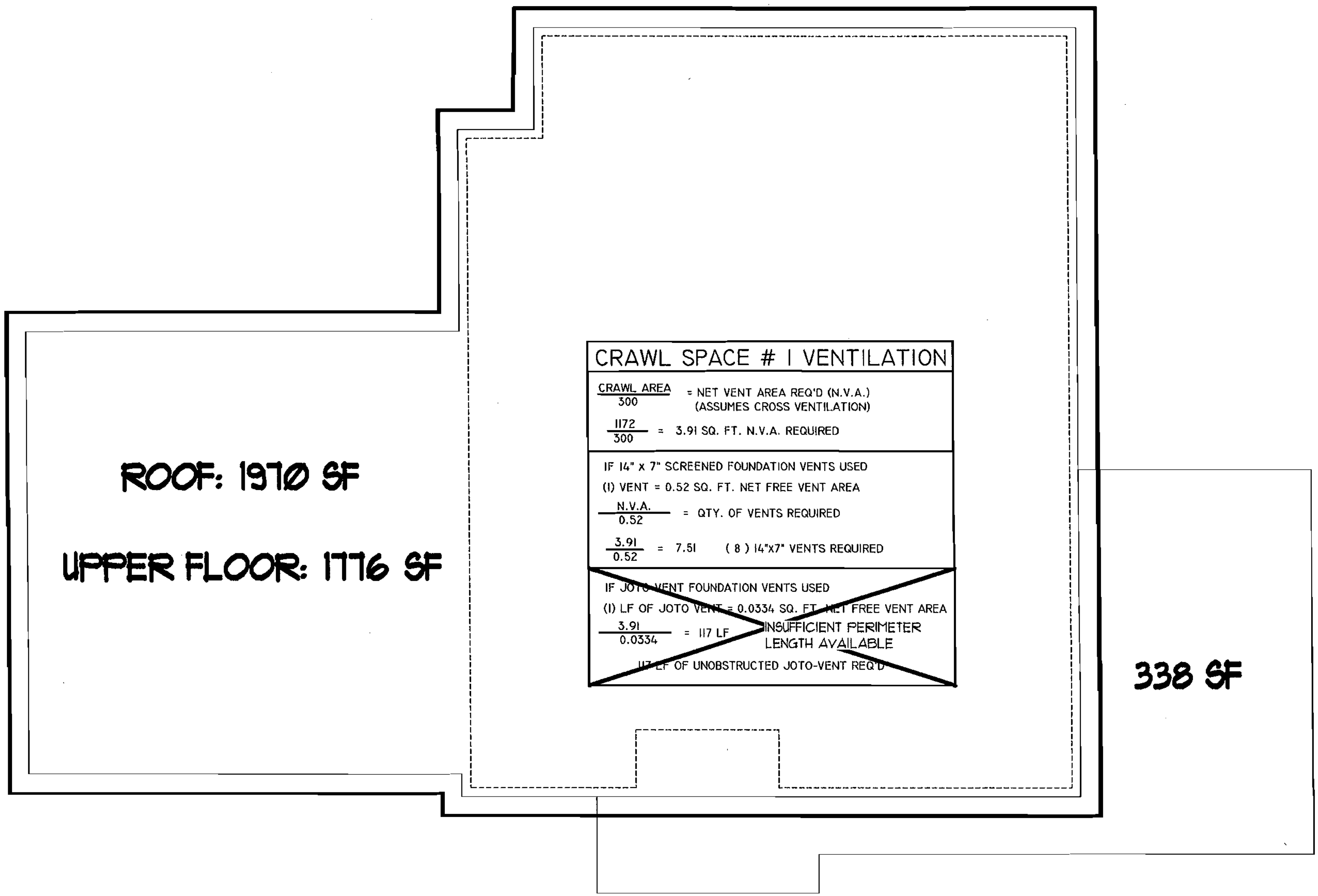
Story Weight at Upper Floor:

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

Story Weight at Main Floor:

$$w_2 := 15 \cdot \text{psf} \cdot A_{2a} + 10 \cdot \text{psf} \cdot A_{2b} + 12 \cdot \text{psf} \cdot (4.5 \cdot \text{ft} \cdot P_1 + 5\text{ft} \cdot P_2)$$

$$W := w_1 + w_2 = 97454.68\,\text{lb}$$


ROOF: 1970 SF
UPPER FLOOR: 1776 SF
CRAWL SPACE # 1 VENTILATION
CRAWL AREA / 300 = NET VENT AREA REQ'D (N.V.A.) (ASSUMES CROSS VENTILATION)
1172 / 300 = 3.91 SQ. FT. N.V.A. REQUIRED
IF 14" x 7" SCREENED FOUNDATION VENTS USED
(1) VENT = 0.52 SQ. FT. NET FREE VENT AREA
N.V.A. / 0.52 = QTY. OF VENTS REQUIRED
3.91 / 0.52 = 7.51 (8) 14"x7" VENTS REQUIRED
IF JOTO-VENT FOUNDATION VENTS USED
(1) LF OF JOTO VENT = 0.0334 SQ. FT. NET FREE VENT AREA
3.91 / 0.0334 = 117 LF
INSUFFICIENT PERIMETER LENGTH AVAILABLE
117 LF OF UNOBSTRUCTED JOTO-VENT REQ'D
338 SF

Approximate Fundamental Period, T_a:

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

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

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

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

$C_s := \dfrac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.14$ $\quad$ Total Base Shear: $V_E := C_s \cdot W = 13943.52\,\text{lb}$

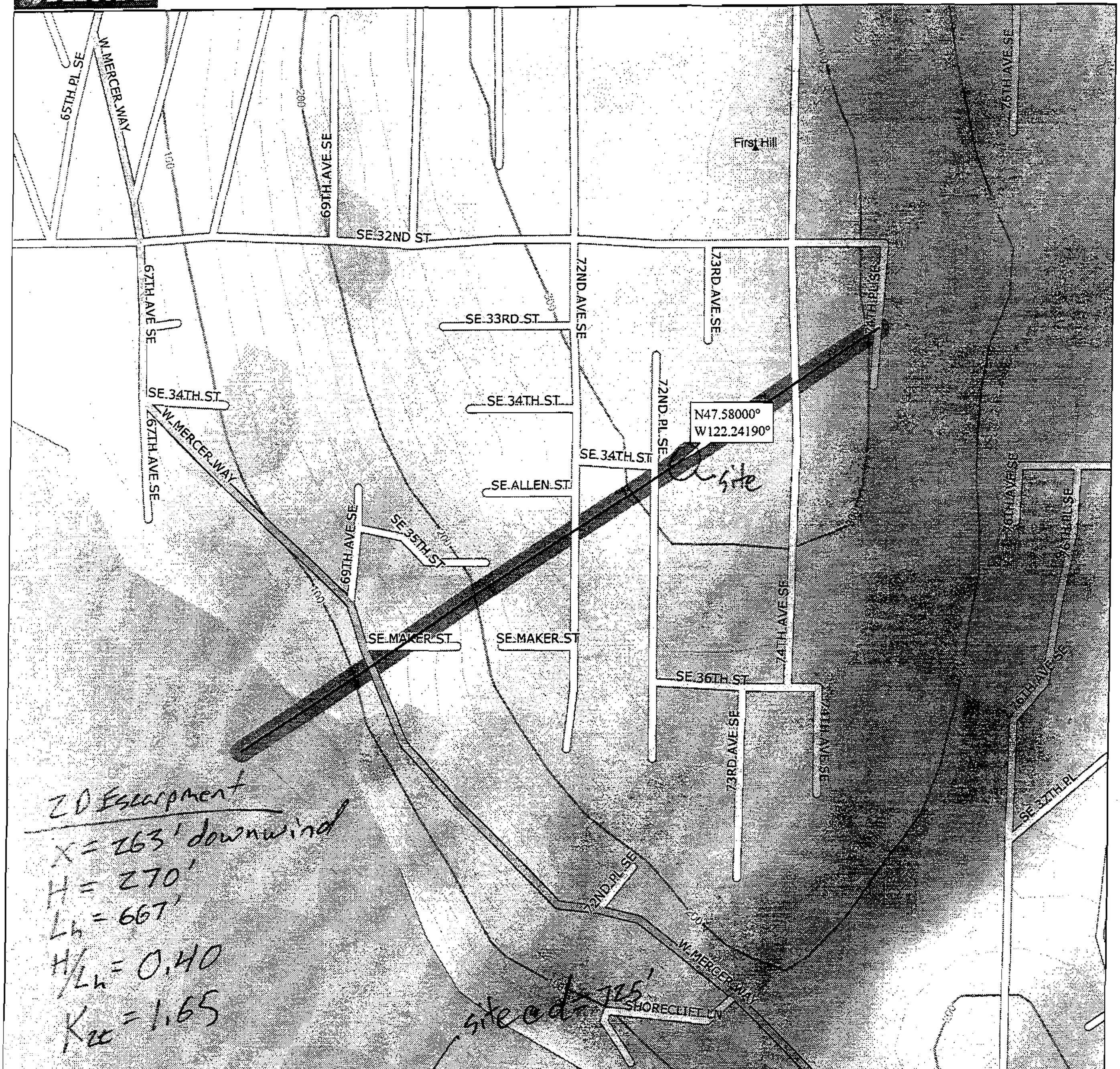
Vertical Shear distribution at each level:

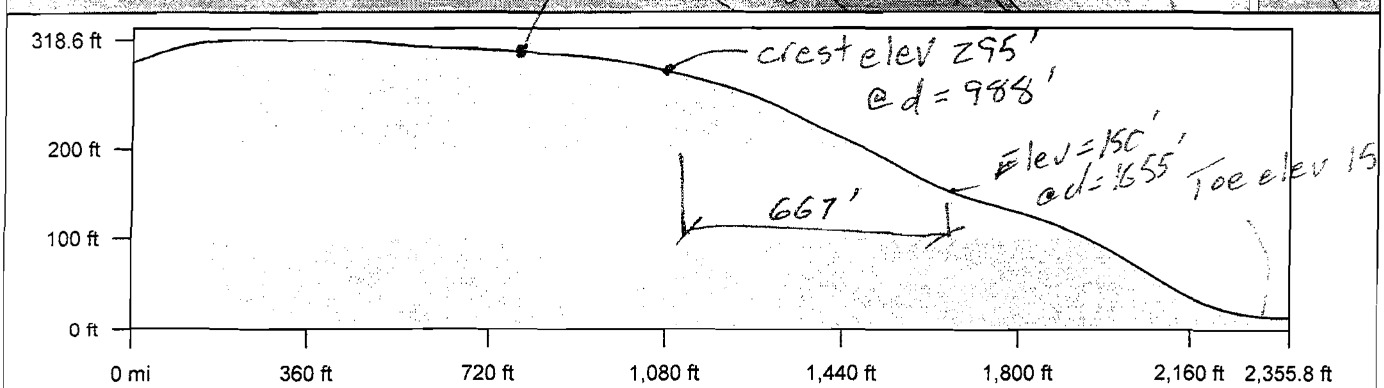
for structures having a period of 0.5 sec or less: $k := 1$

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

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.64$ $\quad F_1 := C_{v1} \cdot V_E = 8912.42\,\text{lb}$ Story Shear at Upper Floor

$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.36$ $\quad F_2 := C_{v2} \cdot V_E = 5031.1\,\text{lb}$ Story Shear at Main Floor





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

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.

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

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

$G := 0.85$ 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)

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

External Pressure Coefficients w/ Roof Pitch = 5/12 (22.6 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.4-1

Front to Back:

$L_{fb} := 40ft$ $B_{fb} := 55ft$ $\frac{L_{fb}}{B_{fb}} = 0.73$ $\frac{h}{L_{fb}} = 0.6$

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

$C_{pf2} := 0.06$ Windward Roof

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

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

Side to Side:

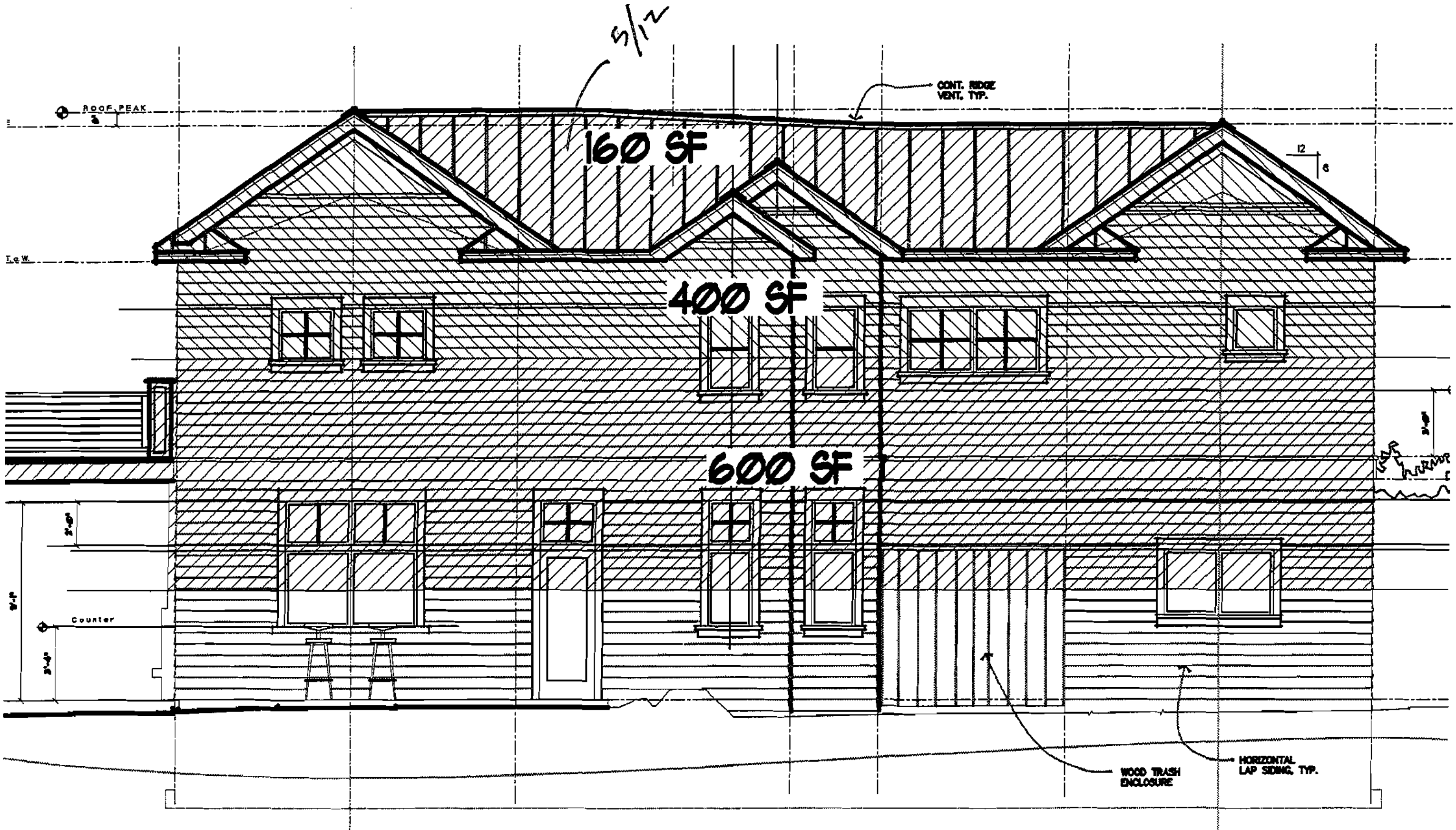
$L_{ss} := 55ft$ $B_{ss} := 40ft$ $\frac{L_{ss}}{B_{ss}} = 1.38$ $\frac{h}{L_{ss}} = 0.44$

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

$C_{ps2} := 0.32$ Windward Roof

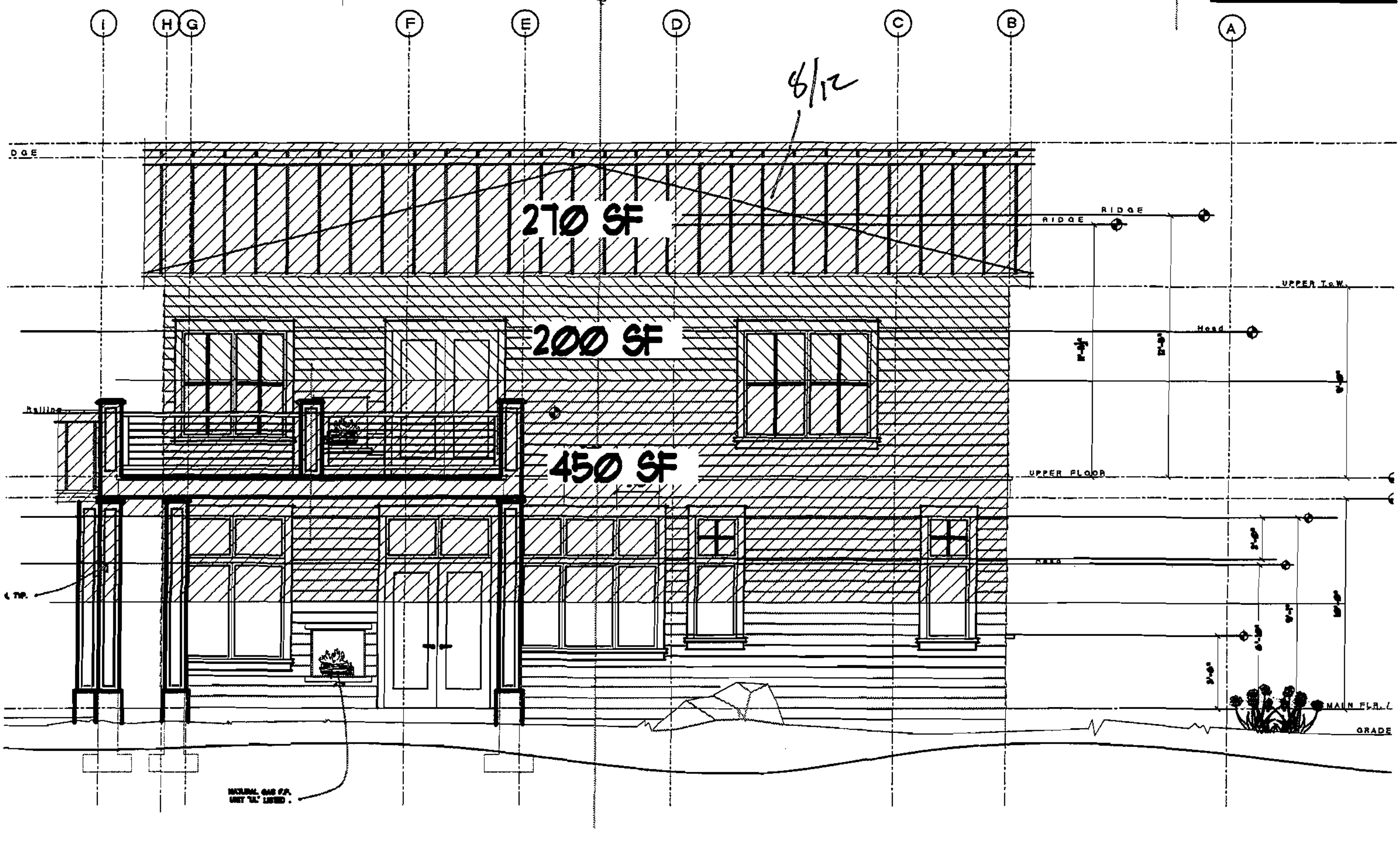
$C_{ps3} := -.6$ Leeward Roof

$C_{ps4} := -.43$ Leeward Wall



5/12
ROOF PEAK
CONT. RIDGE VENT, TYP.
160 SF
T.o.W.
400 SF
600 SF
Counter
WOOD TRASH ENCLOSURE
HORIZONTAL LAP SIDING, TYP.

SOUTH EL



I
H
G
F
E
D
C
B
A
8/12
RIDGE
270 SF
UPPER T.o.W.
Head
200 SF
Railing
450 SF
UPPER FLOOR
MAIN FLR.
GRADE
NATURAL GAS F.P. UNIT 'UL' LISTED.

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

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 2.07\,ft^{-2} \cdot lb$

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

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

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 11.06\,ft^{-2} \cdot lb$

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

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -14.86\,ft^{-2} \cdot lb$

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:

$p_{wr1} - p_{lr1} = 22.8\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw1} = 43.88\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw1} = 42.31\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 31.79\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw2} = 41.46\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw2} = 39.89\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})160ft^2 + (p_{ww1} - p_{lw1}) \cdot 400 \cdot ft^2 = 21198.79\,lb$$

Wind Pressure at Main Floor (Front to Back):

$$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 0ft^2 + (p_{ww1} - p_{lw1}) \cdot 600ft^2 = 26325.29\,lb$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 270ft^2 + (p_{ww1} - p_{lw2}) \cdot 200ft^2 = 16873.86\,lb$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 450ft^2 = 17952.17\,lb$$

Myers Engineering, LLC
3206 50th Street Ct NW, Ste 210-B
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Phone: 253-858-3248
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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 = 34 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)

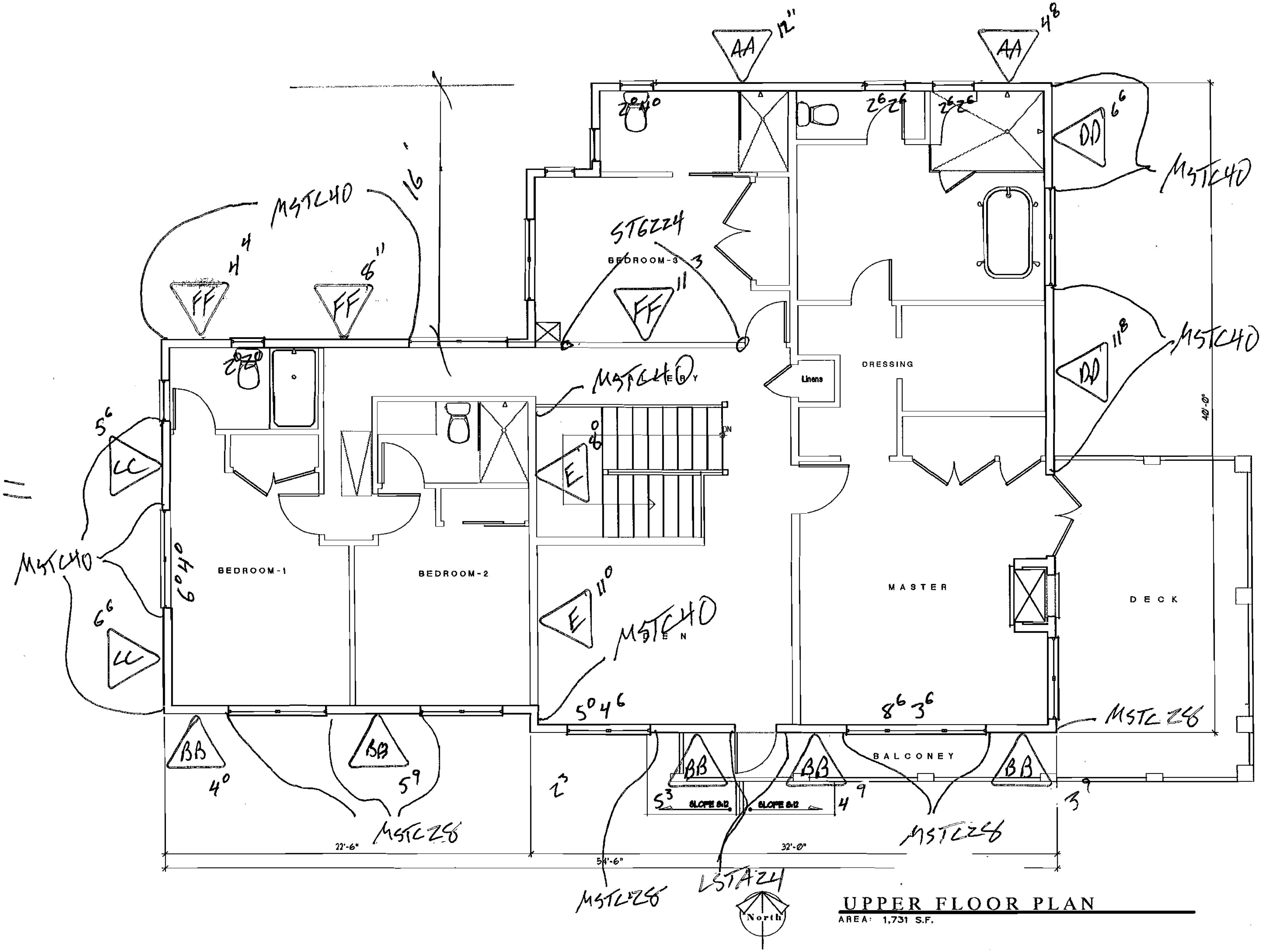
$p_5 := q_h \cdot [(GC_{p5out}) - (GC_{pi})]psf$ $p_5 = -64.22\,ft^{-2} \cdot lb$ (Zone 5)

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

$0.1(40ft) = 4\,ft$ $0.4 \cdot h = 9.6\,ft$ $0.04(40ft) = 1.6\,ft$

Therefore $a := 4ft$



UPPER FLOOR PLAN

AREA: 1,731 S.F.

WALL AA:

Story Shear due to Wind: $V_{3W} = 16873.86\,lb$

Story Shear due to Seismic: $F_1 = 8912.42\,lb$

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

Distance between shear walls: $L_1 := 16 \cdot ft$

Shear Wall Length: $Laa_w := (12.92 + 4.67)ft = 17.59\,ft$ $\quad Laa_s := (12.92 + 4.67)ft = 17.59\,ft$

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

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

$vaa = 115.11\,ft^{-1} \cdot lb \quad \frac{vaa}{C_o} = 115.11\,ft^{-1} \cdot lb$

$E_{aa} = 70.93\,ft^{-1} \cdot lb \quad \frac{E_{aa}}{C_o} = 70.93\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{aa} := 4.67 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRaa := \frac{W_{aa} \cdot L_{aa}}{2} \quad DLRaa = 280.2\,lb$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} \quad CFaa_w = 1036.03\,lb$

$CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} \quad CFaa_s = 638.41\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 867.91\,lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 506.77\,lb$

No Holdowns Required

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

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

$B_p := \frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 1.42\,ft \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 2.3\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vaa} = 11.95\,ft \quad \frac{(Z_B \cdot C_o)}{E_{aa}} = 19.4\,ft$

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

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

Phone: 253-858-3248
Email: myengineer@centurytel.net

WALL BB:

Story Shear due to Wind: $V_{3W} = 16873.86\,lb$ Story Shear due to Seismic: $F_1 = 8912.42\,lb$

Bldg Width in direction of Load: $L_t := 40 \cdot ft$ Distance between shear walls: $L_1 := 24 \cdot ft$

Shear Wall Length: $Lbb_w := (4 + 5.75 + 5.25 + 4.75 + 3.75)ft = 23.5\,ft$ $Lbb_s := \left[4\left(\frac{8}{9}\right) + 5.75 + 5.25 + 4.75 + 3.75\left(\frac{7.5}{9}\right)\right]ft = 22.43\,ft$

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: $vbb := \dfrac{\frac{0.6V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Lbb_w}$ Seismic Force: $\rho := 1.0$ $E_{bb} := \dfrac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1}{2}}{Lbb_s}$

$vbb = 129.25\,lb \cdot ft^{-1}$ $\frac{vbb}{C_o} = 129.25\,lb \cdot ft^{-1}$ $E_{bb} = 83.44\,lb \cdot ft^{-1}$ $\frac{E_{bb}}{C_o} = 83.44\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_{bb} := 3.75 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{bb} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $DLRbb = 225\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_w = 1163.22\,lb$ $CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_s = 750.96\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 1028.22\,lb$ $HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS}) \cdot DLRbb = 645.26\,lb$

Simpson MSTC28 to wall or LSTA24 to beam

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 1.26\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 1.96\,ft$

16d @ 12" o.c.

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

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

$A_s := \frac{(Z_B \cdot C_o)}{vbb} = 10.65\,ft$ $\frac{(Z_B \cdot C_o)}{E_{bb}} = 16.49\,ft$

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 21198.79\,lb$

Story Shear due to Seismic: $F_1 = 8912.42\,lb$

Bldg Width in direction of Load: $L_t := 54.5 \cdot ft$

Distance between shear walls: $L_1 := 22.5 \cdot ft$

Shear Wall Length: $Lcc_w := (6.5 + 5.5)ft = 12\,ft$

$Lcc_s := (6.5 + 5.5)ft = 12\,ft$

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

$vcc = 218.79\,ft^{-1} \cdot lb$ $\quad \frac{vcc}{C_o} = 218.79\,ft^{-1} \cdot lb$

$E_{cc} = 107.32\,ft^{-1} \cdot lb$ $\quad \frac{E_{cc}}{C_o} = 107.32\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 5.5 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRcc := \frac{W_{cc} \cdot L_{cc}}{2}$ $\quad DLRcc = 577.5\,lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $\quad CFcc_w = 1969.15\,lb$

$CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $\quad CFcc_s = 965.85\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 1622.65\,lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 694.54\,lb$

Simpson MSTC40 strap

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 0.75\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 1.52\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vcc} = 6.29\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_{cc}} = 12.82\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 21198.79\,lb$ Story Shear due to Seismic: $F_1 = 8912.42\,lb$

Bldg Width in direction of Load: $L_t := 54.5 \cdot ft$ Distance between shear walls: $L_1 := 32 \cdot ft$

Shear Wall Length: $Ldd_w := (6.5 + 11.67)ft = 18.17\,ft$ $Ldd_s := (6.5 + 11.67)ft = 18.17\,ft$

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

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

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

$vdd = 205.51\,ft^{-1} \cdot lb$ $\frac{vdd}{C_o} = 205.51\,ft^{-1} \cdot lb$ $E_{dd} = 100.8\,ft^{-1} \cdot lb$ $\frac{E_{dd}}{C_o} = 100.8\,ft^{-1} \cdot lb$

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

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

$W_{dd} := (15 \cdot psf) \cdot 9 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRdd := \frac{W_{dd} \cdot L_{dd}}{2}$ $DLRdd = 731.25\,lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_w = 1849.58\,lb$ $CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_s = 907.2\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 1410.83\,lb$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 563.66\,lb$

Simpson MSTC40 to wall or ST6224 to flush beam

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 0.79\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 1.62\,ft$

16d @ 8" o.c.

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

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

$A_S := \frac{(Z_B \cdot C_o)}{vdd} = 6.7\,ft$ $\frac{(Z_B \cdot C_o)}{E_{dd}} = 13.65\,ft$

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

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

Phone: 253-858-3248
Email: myengineer@centurytel.net

WALL EE:

Story Shear due to Wind: $V_{1W} = 21198.79\,lb$ Story Shear due to Seismic: $F_1 = 8912.42\,lb$

Bldg Width in direction of Load: $L_t := 54.5\cdot ft$ Distance between shear walls: $L_1 := 22.5\cdot ft$ $L_2 := 32ft$

Shear Wall Length: $Lee_w := (8 + 11)ft = 19\,ft$ $Lee_s := (8 + 11)ft = 19\,ft$

Percent full height sheathing: $\% := \left(\frac{19\cdot ft}{19.5\cdot ft}\right)\cdot 100$ $\% = 97.44$ Max Opening Height = 9ft-0in, Therefore $C_o := 0.95$ per AF&PA SDPWS Table 4.3.3.5

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

$vee = 334.72\,ft^{-1}\cdot lb$ $\frac{vee}{C_o} = 352.33\,ft^{-1}\cdot lb$ $E_{ee} = 164.18\,ft^{-1}\cdot lb$ $\frac{E_{ee}}{C_o} = 172.82\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{ee} := 8\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{ee} := (15\cdot psf)\cdot 5\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRee := \frac{W_{ee}\cdot L_{ee}}{2}$ $DLRee = 660\,lb$

Chord Force:

$CFee_w := \frac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_w = 3171.01\,lb$ $CFee_s := \frac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_s = 1555.35\,lb$

Holdown Force:

$HDFee_w := CFee_w - 0.6DLRee = 2775.01\,lb$ $HDFee_s := CFee_s - (0.6 - 0.14S_{DS})\cdot DLRee = 1245.28\,lb$

Simpson MSTC40 strap

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vee} = 0.46\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{ee}} = 0.94\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vee} = 3.91\,ft$ $\frac{(Z_B\cdot C_o)}{E_{ee}} = 7.96\,ft$

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

WALL FF:

Story Shear due to Wind: $V_{3W} = 16873.86\,lb$ Story Shear due to Seismic: $F_1 = 8912.42\,lb$

Bldg Width in direction of Load: $L_t := 40 \cdot ft$ Distance between shear walls: $L_1 := 16 \cdot ft$ $L_2 := 24ft$

Shear Wall Length: $Lff_w := (4.33 + 8.92 + 11.25)ft = 24.5\,ft$ $Lff_s := (4.33 + 8.92 + 11.25)ft = 24.5\,ft$

Percent full height sheathing: $\% := \left(\frac{12.25 \cdot ft}{14.25 \cdot ft}\right) \cdot 100$ $\% = 85.96$

Max Opening Height = 2ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vff := \dfrac{\frac{0.6V_{3W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lff_w}$

Seismic Force: $\rho := 1.0$ $E_{ff} := \dfrac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lff_s}$

$vff = 206.62\,ft^{-1} \cdot lb$ $\frac{vff}{C_o} = 206.62\,ft^{-1} \cdot lb$ $E_{ff} = 127.32\,ft^{-1} \cdot lb$ $\frac{E_{ff}}{C_o} = 127.32\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{ff} := 11.25 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{ff} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRff := \frac{W_{ff} \cdot L_{ff}}{2}$ $DLRff = 675\,lb$

Chord Force:

$CFff_w := \frac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $CFff_w = 1859.57\,lb$ $CFff_s := \frac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $CFff_s = 1145.88\,lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6DLRff = 1454.57\,lb$ $HDFff_s := CFff_s - (0.6 - 0.14S_{DS}) \cdot DLRff = 828.77\,lb$

Simpson MSTC40 strap to wall or ST6224 strap to beam

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

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

$B_s := \frac{(C_D \cdot Z_N \cdot C_o)}{vff} = 0.79\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ff}} = 1.28\,ft$

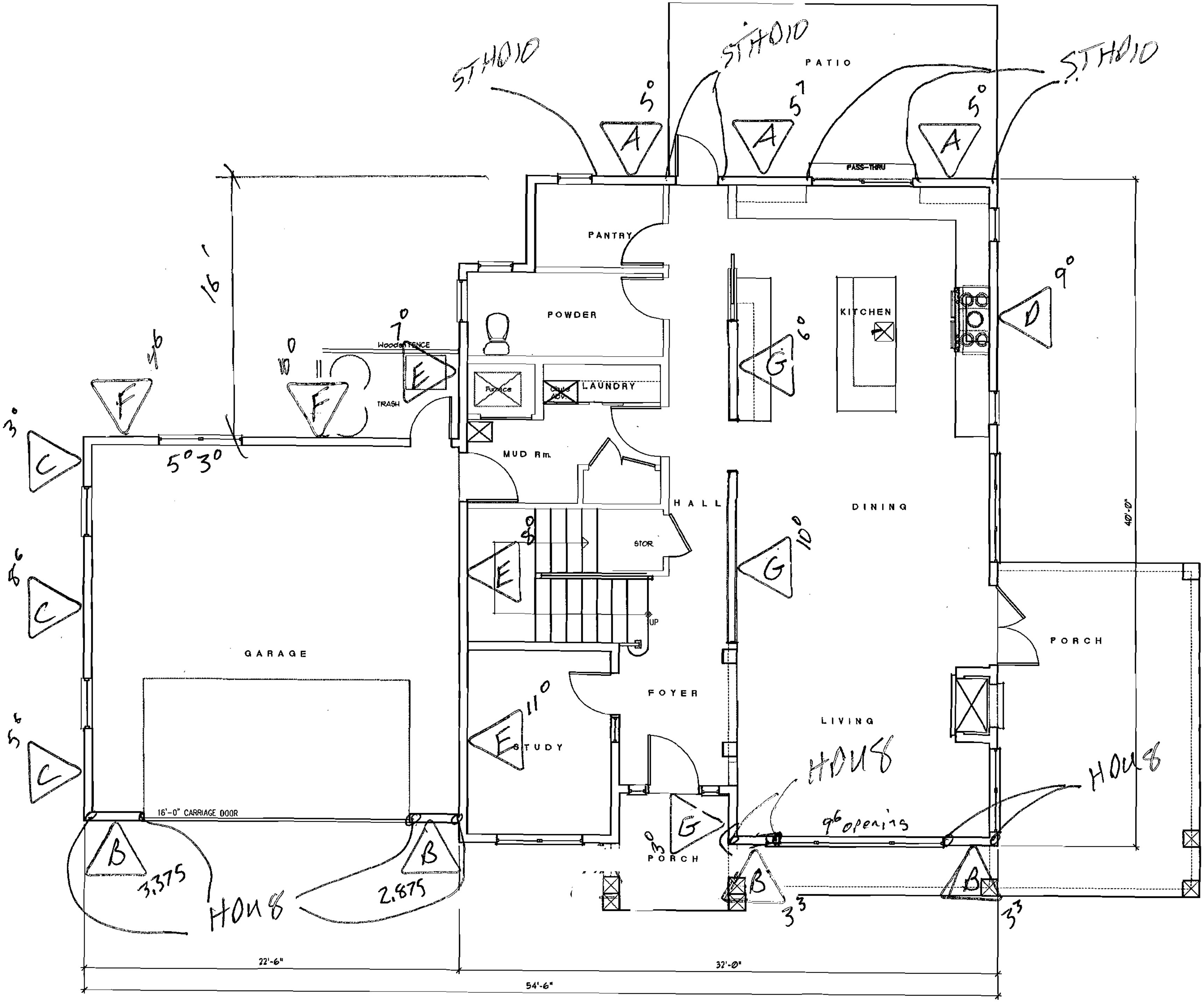
16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vff} = 6.66\,ft$ $\frac{(Z_B \cdot C_o)}{E_{ff}} = 10.81\,ft$

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



MAIN FLOOR PLAN

AREA: 1,718 S.F.

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

Phone: 253-858-3248
Email: myengineer@centurytel.net

PROJECT : **3402 72nd Place SE**

WALL A:

Story Shear due to Wind: $V_{4W} = 17952.17\,\mathrm{lb}$ Story Shear due to Seismic: $F_2 = 5031.1\,\mathrm{lb}$

Bldg Width in direction of Load: $L_t := 40\cdot\mathrm{ft}$ Distance between shear walls: $L_1 := 16\cdot\mathrm{ft}$

Shear Wall Length: $La_w := (2\cdot5 + 5.58)\mathrm{ft} = 15.58\,\mathrm{ft}$ $La_s := (2\cdot5 + 5.58)\mathrm{ft} = 15.58\,\mathrm{ft}$

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

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

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

$va = 268.24\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$ $\frac{va}{C_o} = 268.24\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$ $E_a = 125.29\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$ $\frac{E_a}{C_o} = 125.29\,\mathrm{ft}^{-1}\cdot\mathrm{lb}$

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

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

$W_a := (15\cdot\mathrm{psf})\cdot0\cdot\mathrm{ft} + (10\cdot\mathrm{psf})\cdot Pt + (10\mathrm{psf})\cdot1\mathrm{ft}$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 275\,\mathrm{lb}$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 2682.36\,\mathrm{lb}$ $CFa_w + CFaa_w = 3718.39\,\mathrm{lb}$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 1252.95\,\mathrm{lb}$ $CFa_s + CFaa_s = 1891.36\,\mathrm{lb}$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 2517.36\,\mathrm{lb}$ $HDFa_w + HDFaa_w = 3385.27\,\mathrm{lb}$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 1123.75\,\mathrm{lb}$ $HDFa_s + HDFaa_s = 1630.53\,\mathrm{lb}$

Simpson STHD10

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.61\,\mathrm{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 1.3\,\mathrm{ft}$

16d @ 6" o.c.

Anchor Bolt Spacing (NDS TR12 Calcs. w/ 3/4" gap)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 590\cdot\mathrm{lb}$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 944\,\mathrm{lb}$

$As := \frac{(Z_B\cdot C_o)}{va} = 3.52\,\mathrm{ft}$ $\frac{(Z_B\cdot C_o)}{E_a} = 7.53\,\mathrm{ft}$

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

WALL B:

Story Shear due to Wind: $V_{4W} = 17952.17\,lb$ Story Shear due to Seismic: $F_2 = 5031.1\,lb$

Bldg Width in direction of Load: $L_t := 40\cdot ft$ Distance between shear walls: $L_1 := 24\cdot ft$

Shear Wall Length:

$Lb_w := (3.375 + 2.875 + 2\cdot 3.25)ft = 12.75\,ft$

$$Lb_s := \left[3.375\left(\frac{6.75}{10}\right) + 2.875\left(\frac{5.75}{10}\right) + 2\cdot 3.25\left(\frac{6.5}{10}\right)\right]ft = 8.16\,ft$$

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

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

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

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

$vb = 491.66\,lb\cdot ft^{-1}$ $\frac{vb}{C_o} = 491.66\,lb\cdot ft^{-1}$ $E_b = 359.01\,lb\cdot ft^{-1}$ $\frac{E_b}{C_o} = 359.01\,lb\cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_b := 2.875\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_b := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$ $DLRb := \frac{W_b\cdot L_b}{2}$ $DLRb = 201.25\,lb$

Chord Force:

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_w = 4916.62\,lb$ $CFb_w + CFbb_w = 6079.84\,lb$

$CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_s = 3590.05\,lb$ $CFb_s + CFbb_s = 4341.02\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 4795.87\,lb$

$HDFb_w + HDFbb_w = 5824.09\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14S_{DS})\cdot DLRb = 3495.51\,lb$

$HDFb_s + HDFbb_s = 4140.76\,lb$

Simpson HDU8 w/ SB7/8x24 anchor

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 0.33\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.45\,ft$

16d @ 4" o.c.

Anchor Bolt Spacing (NDS TR12 Calcs. w/ 3/4" gap)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 590\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 944\,lb$

$As := \frac{(Z_B\cdot C_o)}{vb} = 1.92\,ft$ $\frac{(Z_B\cdot C_o)}{E_b} = 2.63\,ft$

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

WALL C:

Story Shear due to Wind: $V_{2W} = 26325.29\,\text{lb}$

Story Shear due to Seismic: $F_2 = 5031.1\,\text{lb}$

Bldg Width in direction of Load: $L_t := 54.5\cdot\text{ft}$

Distance between shear walls: $L_1 := 22.5\cdot\text{ft}$

Shear Wall Length: $Lc_w := (5.5 + 8.5 + 3)\text{ft} = 17\,\text{ft}$

$$Lc_s := \left[5.5 + 8.5 + 3\left(\frac{6}{10}\right)\right]\text{ft} = 15.8\,\text{ft}$$

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

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

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

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

$vc = 346.24\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{vc}{C_o} = 346.24\,\text{ft}^{-1}\cdot\text{lb}$

$E_c = 127.52\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{E_c}{C_o} = 127.52\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_c := 3\cdot\text{ft}$ $\quad$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_c := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 11\text{ft}$

$DLRc := \frac{W_c\cdot L_c}{2}$ $\quad DLRc = 315\,\text{lb}$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_w = 3462.36\,\text{lb}$ $\quad CFc_w + CFcc_w = 5431.51\,\text{lb}$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_s = 1275.17\,\text{lb}$ $\quad CFc_s + CFcc_s = 2241.03\,\text{lb}$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 3273.36\,\text{lb}$

$HDFc_w + HDFcc_w = 4896.01\,\text{lb}$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 1127.19\,\text{lb}$

$HDFc_s + HDFcc_s = 1821.73\,\text{lb}$

Simpson STHD14

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 0.47\,\text{ft}$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 1.28\,\text{ft}$

16d @ 6" o.c.

Anchor Bolt Spacing (NDS TR12 Calcs. w/ 3/4" gap)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 590\cdot\text{lb}$ $\quad C_D := 1.6$ $\quad Z_B := A_s\cdot C_D$ $\quad Z_B = 944\,\text{lb}$

$As := \frac{(Z_B\cdot C_o)}{vc} = 2.73\,\text{ft}$ $\quad \frac{(Z_B\cdot C_o)}{E_c} = 7.4\,\text{ft}$

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

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

PROJECT : **3402 72nd Place SE**

Phone: 253-858-3248
Email: myengineer@centurytel.net

WALL D:

Story Shear due to Wind: $V_{2W} = 26325.29\,lb$

Story Shear due to Seismic: $F_2 = 5031.1\,lb$

Bldg Width in direction of Load: $L_t := 54.5\cdot ft$

Distance between shear walls: $L_1 := 16\cdot ft$

Shear Wall Length: $Ld_w := (9)ft = 9\,ft$ $Ld_s := (9)ft = 9\,ft$

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

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

$vd = 672.52\,ft^{-1}\cdot lb$ $\frac{vd}{C_o} = 672.52\,ft^{-1}\cdot lb$

$E_d = 260.94\,ft^{-1}\cdot lb$ $\frac{E_d}{C_o} = 260.94\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_d := 9\cdot ft$ Plate Height: $Pt := 10\cdot ft$

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

$DLRd := \frac{W_d\cdot L_d}{2}$ $DLRd = 810\,lb$

Chord Force:

$CFd_w := \frac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_w = 6725.17\,lb$
$CFd_w + CFdd_w = 8574.75\,lb$

$CFd_s := \frac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_s = 2609.44\,lb$
$CFd_s + CFdd_s = 3516.65\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 6239.17\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 2228.91\,lb$

SImpson HDU8 w/ SB7/8x24 Anchor

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

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

$B_o := \frac{(C_D\cdot Z_N\cdot C_o)}{vd} = 0.24\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 0.63\,ft$

16d @ 3" o.c.

Anchor Bolt Spacing (NDS TR12 Calcs. w/ 3/4" gap)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 590\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 944\,lb$

$As := \frac{(Z_B\cdot C_o)}{vd} = 1.4\,ft$ $\frac{(Z_B\cdot C_o)}{E_d} = 3.62\,ft$

5/8" A.B. @ 16" o.c.

WALL E:

Story Shear due to Wind: $V_{2W} = 26325.29\,lb$ Story Shear due to Seismic: $F_2 = 5031.1\,lb$

Bldg Width in direction of Load: $L_t := 54.5\cdot ft$ Distance between shear walls: $L_1 := 22.5\cdot ft$ $L_2 := 16ft$

Shear Wall Length: $Le_w := (7+8+11)ft = 26\,ft$ $Le_s := (7+8+11)ft = 26\,ft$

Percent full height sheathing: $\% := \left(\frac{19\cdot ft}{19.5\cdot ft}\right)\cdot 100\ \% = 97.44$ Max Opening Height = 10ft-0in, Therefore $C_o := 0.95$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $ve := \dfrac{vee\cdot Lee_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1+L_2}{2}\right)}{Le_w}$ Seismic Force: $\rho := 1.0$ $E_e := \dfrac{E_{ee}\cdot Lee_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1+L_2}{2}\right)}{Le_s}$

$ve = 459.18\,ft^{-1}\cdot lb$ $\frac{ve}{C_o} = 483.35\,ft^{-1}\cdot lb$ $E_e = 167.82\,ft^{-1}\cdot lb$ $\frac{E_e}{C_o} = 176.65\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_e := 7\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_e := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 8ft$ $DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 630\,lb$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 4833.47\,lb$ $CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 1766.51\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6DLRe = 4455.47\,lb$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 1470.53\,lb$

Simpson STHD14

$HDFe_w + HDFee_w = 7230.48\,lb$ $HDFe_s + HDFee_s = 2715.82\,lb$

Simpson HDU8 w/ SB7/8x24 anchor

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.34\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.92\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{ve} = 2.85\,ft$ $\frac{(Z_B\cdot C_o)}{E_e} = 7.79\,ft$

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

WALL F:

Story Shear due to Wind: $V_{4W} = 17952.17\,lb$ Story Shear due to Seismic: $F_2 = 5031.1\,lb$

Bldg Width in direction of Load: $L_t := 40 \cdot ft$ Distance between shear walls: $L_1 := 16 \cdot ft$ $L_2 := 24ft$

Shear Wall Length: $Lf_w := (4.5 + 10)ft = 14.5\,ft$ $Lf_s := \left[4.5\left(\frac{9}{10}\right) + 10\right]ft = 14.05\,ft$

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

Wind Force: $vf := \dfrac{vff \cdot Lff_w + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lf_w}$ Seismic Force: $\rho := 1.0$ $E_f := \dfrac{E_{ff} \cdot Lff_s + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lf_s}$

$vf = 720.54\,ft^{-1} \cdot lb$ $\frac{vf}{C_o} = 720.54\,ft^{-1} \cdot lb$ $E_f = 347.35\,ft^{-1} \cdot lb$ $\frac{E_f}{C_o} = 347.35\,ft^{-1} \cdot lb$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 896 plf
Seismic Capacity = 640 plf

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

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 247.5\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 7205.39\,lb$ $CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 3473.47\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 7056.89\,lb$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 3357.2\,lb$

Simpson HDU8 at 4x6 postw/ SB7/8x24 anchor

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 0.23\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 0.47\,ft$

16d @ 3" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vf} = 1.91\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 3.96\,ft$

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

WALL G:

Story Shear due to Wind: $V_{2W} = 26325.29\,lb$

Story Shear due to Seismic: $F_2 = 5031.1\,lb$

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

Distance between shear walls: $L_1 := 16\cdot ft$ $L_2 := 16ft$

Shear Wall Length: $Lg_w := (3 + 6 + 10)ft = 19\,ft$

$Lg_s := \left[3\left(\frac{6}{10}\right) + 6 + 10\right]ft = 17.8\,ft$

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

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

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

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

$vg = 246.32\,ft^{-1}\cdot lb$ $\frac{vg}{C_o} = 246.32\,ft^{-1}\cdot lb$

$E_g = 58.62\,ft^{-1}\cdot lb$ $\frac{E_g}{C_o} = 58.62\,ft^{-1}\cdot lb$

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

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

$W_g := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 12ft$

$DLRg := \frac{W_g\cdot L_g}{2}$ $DLRg = 330\,lb$

Chord Force:

$CFg_w := \frac{vg\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_w = 2463.19\,lb$

$CFg_s := \frac{E_g\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_s = 586.23\,lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6\cdot DLRg = 2265.19\,lb$

$HDFg_s := CFg_s - (0.6 - 0.14S_{DS})\cdot DLRg = 431.19\,lb$

Simpson LSTHD8 or HDU2 w/ PAB5 anchor or epoxied anchor w/ ~~8"~~ 6" embed in 24" wide footing

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vg} = 0.66\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_g} = 2.78\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vg} = 5.59\,ft$ $\frac{(Z_B\cdot C_o)}{E_g} = 23.47\,ft$

5/8" A.B. @ 66" o.c.

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa\cdot\frac{Laa_w}{28ft} = 72.32\,ft^{-1}\cdot lb \qquad E_{aa}\cdot\frac{Laa_s}{28ft} = 44.56\,ft^{-1}\cdot lb$$

Wall Lines BB:

$$vbb\cdot\frac{Lbb_w}{54ft} = 56.25\,ft^{-1}\cdot lb \qquad E_{bb}\cdot\frac{Lbb_s}{54ft} = 34.66\,ft^{-1}\cdot lb$$

Wall Lines CC:

$$vcc\cdot\frac{Lcc_w}{23ft} = 114.15\,ft^{-1}\cdot lb \qquad E_{cc}\cdot\frac{Lcc_s}{23ft} = 55.99\,ft^{-1}\cdot lb$$

Wall Lines DD:

$$vdd\cdot\frac{Ldd_w}{40ft} = 93.35\,ft^{-1}\cdot lb \qquad E_{dd}\cdot\frac{Ldd_s}{40ft} = 45.79\,ft^{-1}\cdot lb$$

Wall Lines EE:

$$vee\cdot\frac{Lee_w}{34ft} = 187.05\,ft^{-1}\cdot lb \qquad E_{ee}\cdot\frac{Lee_s}{34ft} = 91.75\,ft^{-1}\cdot lb$$

Wall Lines FF:

$$vff\cdot\frac{Lff_w}{54ft} = 93.74\,ft^{-1}\cdot lb \qquad E_{ff}\cdot\frac{Lff_s}{54ft} = 57.77\,ft^{-1}\cdot lb$$

Floor Diaphragm load

Rim to Top Plate Connection

Wall Lines A:

$$\frac{va\cdot La_w - vaa\cdot Laa_w}{28ft} = 76.94\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La_s - E_{aa}\cdot Laa_s}{28ft} = 25.16\,ft^{-1}\cdot lb \qquad \frac{va\cdot La_w}{28ft} = 149.25\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La_s}{28ft} = 69.72\,ft^{-1}\cdot lb$$

Wall Lines B:

$$\frac{vb\cdot Lb_w - vbb\cdot Lbb_w}{54ft} = 59.84\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb_s - E_{bb}\cdot Lbb_s}{54ft} = 19.57\,ft^{-1}\cdot lb \qquad \frac{vb\cdot Lb_w}{54ft} = 116.09\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb_s}{54ft} = 54.22\,ft^{-1}\cdot lb$$

Wall Lines C:

$$\frac{vc\cdot Lc_w - vcc\cdot Lcc_w}{23ft} = 141.76\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc_s - E_{cc}\cdot Lcc_s}{23ft} = 31.61\,ft^{-1}\cdot lb \qquad \frac{vc\cdot Lc_w}{23ft} = 255.91\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc_s}{23ft} = 87.6\,ft^{-1}\cdot lb$$

Wall Lines D:

$$\frac{vd\cdot Ld_w - vdd\cdot Ldd_w}{40ft} = 57.96\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld_s - E_{dd}\cdot Ldd_s}{40ft} = 12.92\,ft^{-1}\cdot lb \qquad \frac{vd\cdot Ld_w}{40ft} = 151.32\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld_s}{40ft} = 58.71\,ft^{-1}\cdot lb$$

Wall Lines E:

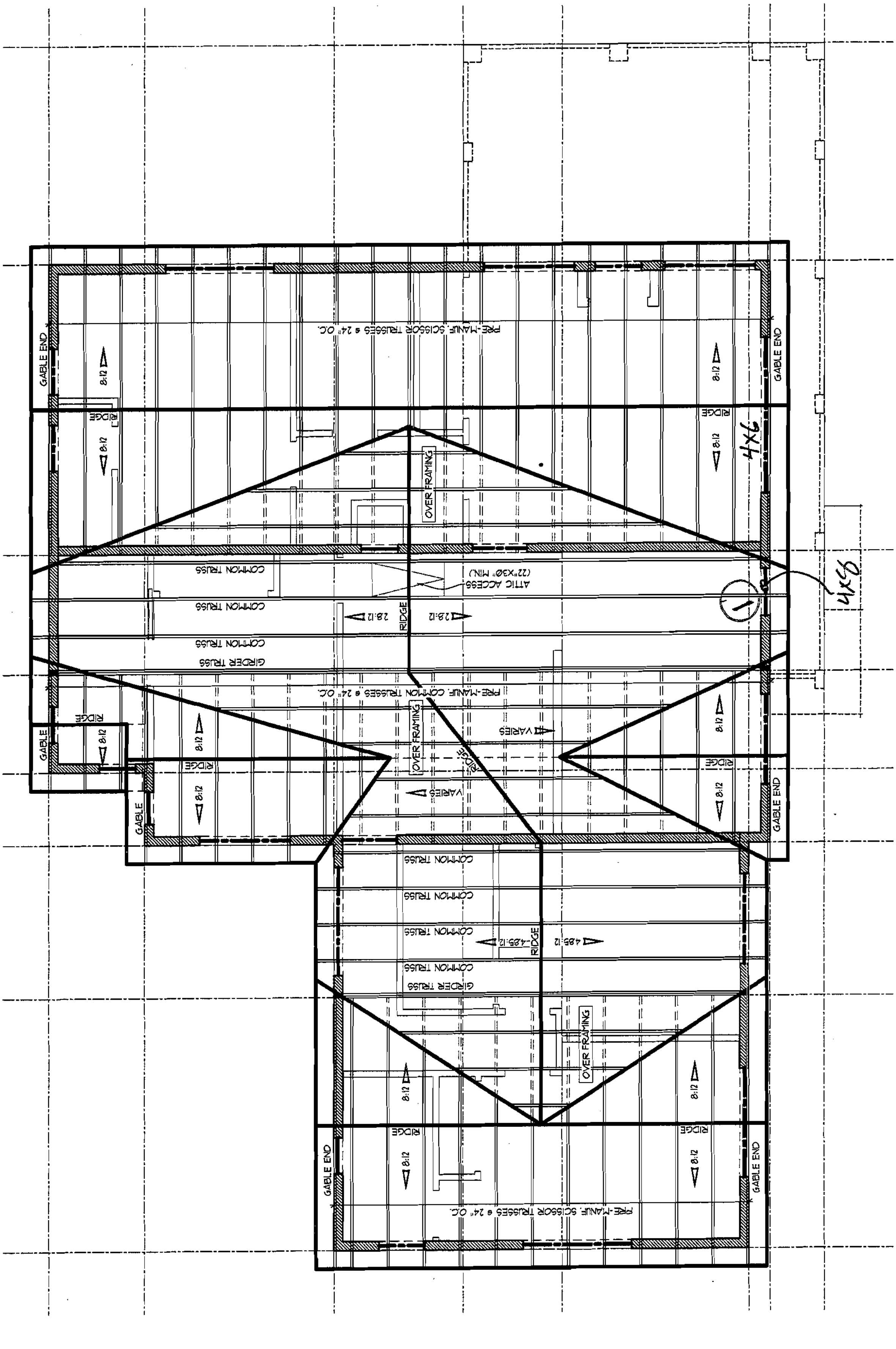
$$\frac{ve\cdot Le_w - vee\cdot Lee_w}{34ft} = 164.09\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le_s - E_{ee}\cdot Lee_s}{34ft} = 36.59\,ft^{-1}\cdot lb \qquad \frac{ve\cdot Le_w}{34ft} = 351.14\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le_s}{34ft} = 128.33\,ft^{-1}\cdot lb$$

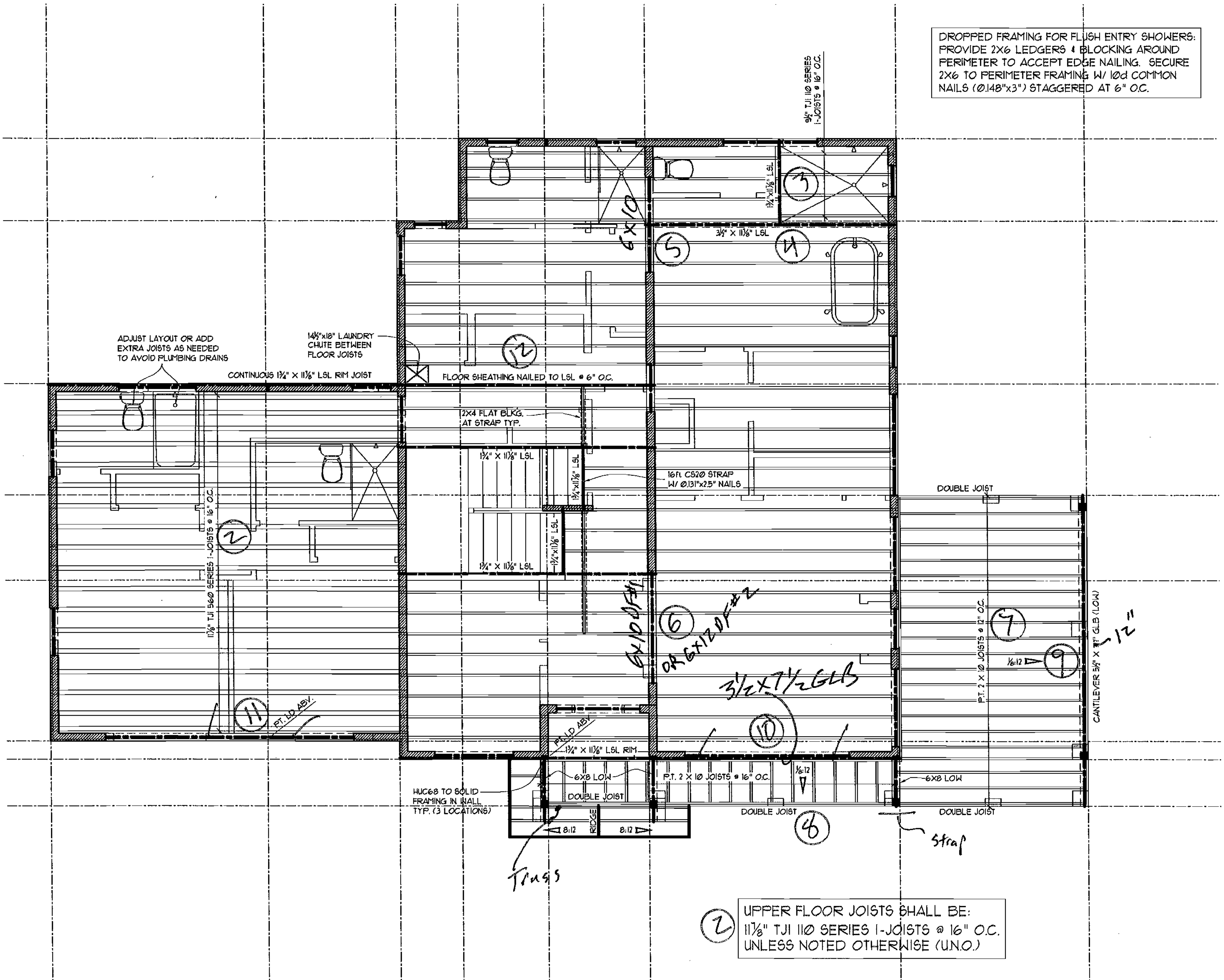
Wall Lines F:

$$\frac{vf\cdot Lf_w - vff\cdot Lff_w}{54ft} = 99.73\,ft^{-1}\cdot lb \qquad \frac{E_f\cdot Lf_s - E_{ff}\cdot Lff_s}{54ft} = 32.61\,ft^{-1}\cdot lb \qquad \frac{vf\cdot Lf_w}{54ft} = 193.48\,ft^{-1}\cdot lb \qquad \frac{E_f\cdot Lf_s}{54ft} = 90.37\,ft^{-1}\cdot lb$$

Wall Lines G:

$$vg\cdot\frac{Lg_w}{40ft} = 117\,ft^{-1}\cdot lb \qquad E_g\cdot\frac{Lg_s}{40ft} = 26.09\,ft^{-1}\cdot lb$$


PRE-MANUF. SCISSOR TRUSSES @ 24" O.C.
GABLE END
8:12
RIDGE
OVER FRAMING
4x6
COMMON TRUSS
ATTIC ACCESS
(22"X30" MIN.)
2.8:12
GIRDER TRUSS
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
VARIES
GABLE
4.85:12


DROPPED FRAMING FOR FLUSH ENTRY SHOWERS: PROVIDE 2X6 LEDGERS & BLOCKING AROUND PERIMETER TO ACCEPT EDGE NAILING. SECURE 2X6 TO PERIMETER FRAMING W/ 10d COMMON NAILS (0.148"x3") STAGGERED AT 6" O.C.
9½" TJI 110 SERIES I-JOISTS @ 16" O.C.
1¾"x11⅞" LSL
3½" X 11⅞" LSL
6X10
ADJUST LAYOUT OR ADD EXTRA JOISTS AS NEEDED TO AVOID PLUMBING DRAINS
14½"x18" LAUNDRY CHUTE BETWEEN FLOOR JOISTS
CONTINUOUS 1¾" X 11⅞" LSL RIM JOIST
FLOOR SHEATHING NAILED TO LSL @ 6" O.C.
2X4 FLAT BLKG. AT STRAP TYP.
1¾" X 11⅞" LSL
16ft CS20 STRAP W/ 0.131"x2.5" NAILS
11⅞" TJI 560 SERIES I-JOISTS @ 16" O.C.
DOUBLE JOIST
6X10 DF#1
OR 6X12 DF#2
3½X7½GLB
P.T. 2 X 10 JOISTS @ 12" O.C.
1/8:12
CANTILEVER 5½" X 7⅞" GLB (LOW)
12"
PT. LD ABV.
P.T. LD ABV.
1¾" X 11⅞" LSL RIM
6X8 LOW
P.T. 2 X 10 JOISTS @ 16" O.C.
HUC68 TO SOLID FRAMING IN WALL TYP. (3 LOCATIONS)
RIDGE
8:12
Truss
Strap
UPPER FLOOR JOISTS SHALL BE: 11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C. UNLESS NOTED OTHERWISE (U.N.O.)

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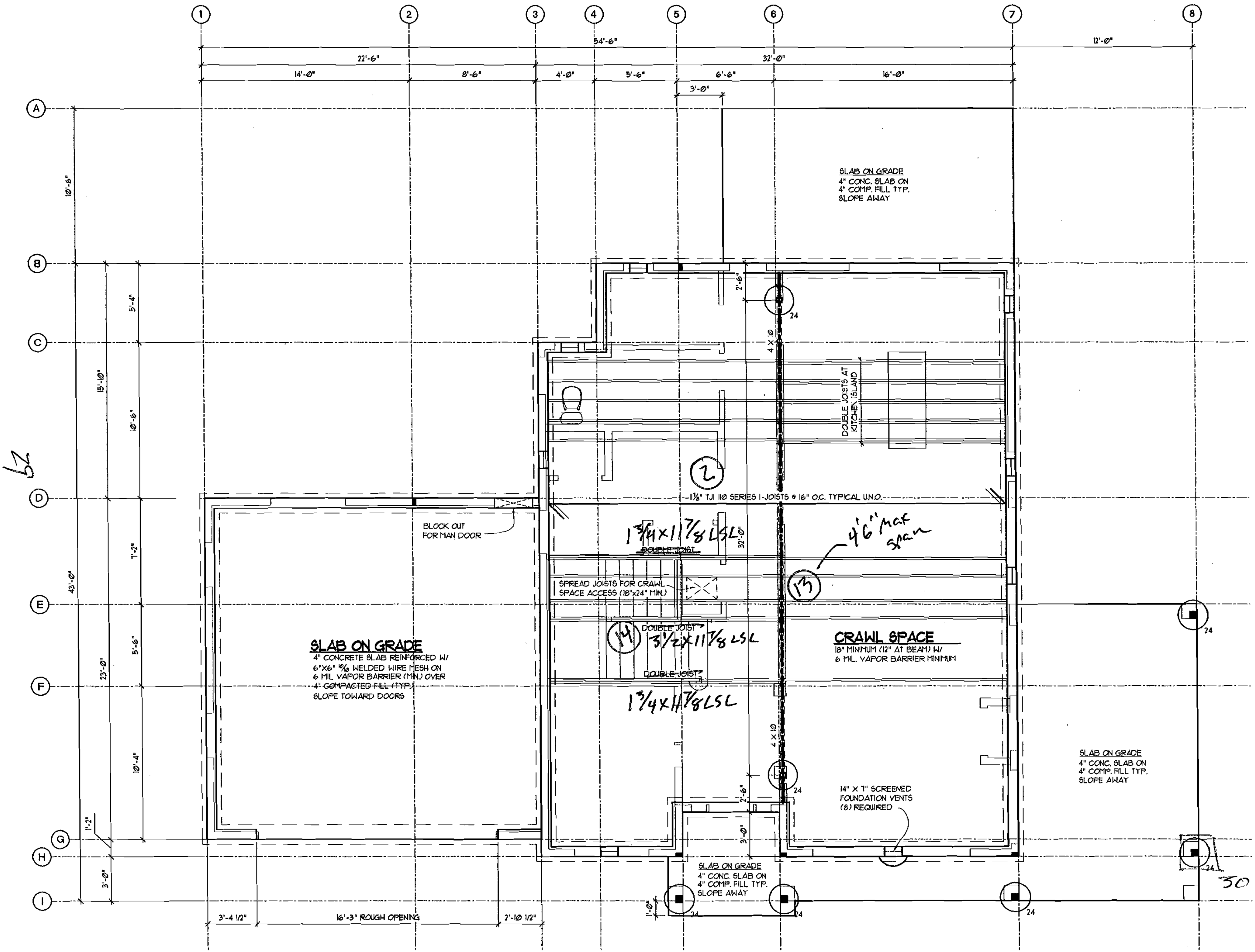

54'-6"
22'-6"
32'-0"
12'-0"
14'-0"
8'-6"
4'-0"
5'-6"
6'-6"
16'-0"
3'-0"
10'-6"
15'-10"
5'-4"
43'-0"
23'-0"
7'-2"
10'-4"
1'-2"
3'-0"
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
4 X 10
24
DOUBLE JOISTS AT
KITCHEN ISLAND
11 7/8" TJI 110 SERIES I-JOISTS @ 16" O.C. TYPICAL U.N.O.
BLOCK OUT
FOR MAN DOOR
1 3/4 x 11 7/8 LSL
DOUBLE JOIST
32'-0"
4'6" max span
SPREAD JOISTS FOR CRAWL
SPACE ACCESS (18"x24" MIN.)
DOUBLE JOIST
3 1/2 x 11 7/8 LSL
DOUBLE JOIST
1 3/4 x 11 7/8 LSL
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
CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/
6 MIL. VAPOR BARRIER MINIMUM
14" X 7" SCREENED
FOUNDATION VENTS
(8) REQUIRED
2'-6"
1'-0"
3'-4 1/2"
16'-3" ROUGH OPENING
2'-10 1/2"
62
50

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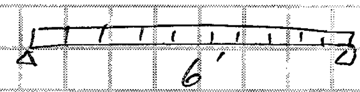
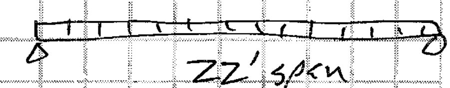
Myers Engineering LLC
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
Fax (253) 858-3249
myengineer@centurytel.net

(1) $w_D = 15psf(42'/2) = 315plf$
$w_S = 25psf(42'/2) = 525plf$



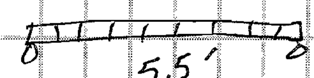
4x10
(4x8 @ 5'8" span)

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



11 7/8 TJI 560 @ 16" o.c.
(TJI 110 @ 17'8" span)

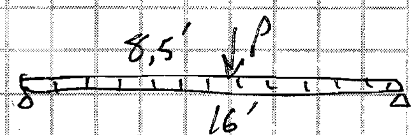
(3) $w_D = 15psf(16'/2) = 120plf$
$w_L = 40psf(16'/2) = 320plf$



1 3/4 x 11 7/8 LSL

(4) $w_D = 15plf$
$w_L = 40plf$

P = 330# DL + 880# LL from (3)

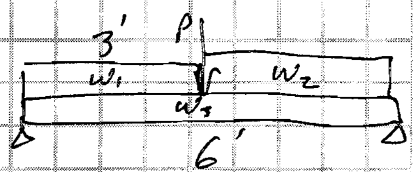


1 3/4 x 11 7/8 LSL

(5) $w_{D_1} = 15psf(20'/2) = 150plf$
$w_{L_1} = 40psf(20'/2) = 400plf$
$w_{D_2} = 15psf(32'/2) = 240plf$
$w_{L_2} = 40psf(32'/2) = 640plf$

$w_{D_3} = 15psf(16'/2) = 120plf$
$w_{S_3} = 25psf(16'/2) = 200plf$

P = 275# DL + 735# LL from (4)



6x10 Minimum

FOR 3402 72nd PL
JOB
DATE 11-23-20
BY MM

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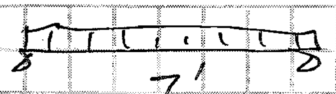
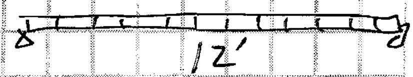
Myers Engineering LLC
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
Fax (253) 858-3249
myengineer@centurytel.net

(6) $w_D = 15psf(32'/2 + 16'/2) = 360plf$
$w_L = 40psf(32'/2) = 640plf$
$w_S = 25psf(16'/2) = 200plf$



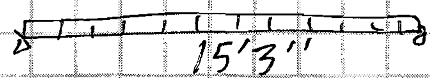
6x10 DF#1
or
6x12 DF#2

(7) $w_D = 10psf$
$w_L = 60psf$
$w_S = 25psf$



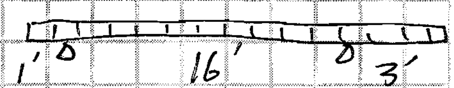
P.T. 2x10 @ 12" o.c.

(8) $w_D = 10psf(3'/2) = 15plf$
$w_L = 60psf(3'/2) = 90plf$
$w_S = 25psf(3'/2) = 37.5plf$



P.T. (2) 2x10

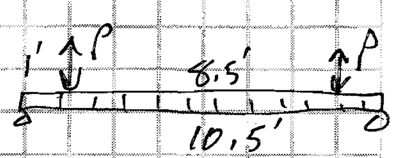
(9) $w_D = 10psf(12'/2) = 60plf$
$w_L = 60psf(12'/2) = 360plf$
$w_S = 25psf(12'/2) = 150plf$



5½x12 GLB

(10) $w_D = 15psf(1' + 2') + 12psf(9') = 153plf$
$w_L = 40plf + 60psf(3'/2) = 130plf$
$w_S = 25psf(2') = 50plf$

$P = \pm 1165^{\#} WL \pm 750^{\#} EL$ from BB $w/\Omega = 3.0$



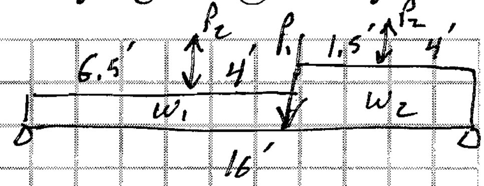
3½x7½ GLB

FOR 3402 72nd PL
JOB
DATE 11-23-20
BY [signature]

MYERS ENGINEERING

Myers Engineering LLC
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
Fax (253) 858-3249
myengineer@centurytel.net

(11) $w_{D_1} = 15psf(2'+1') + 12psf(9') = 153plf$
$w_{L_1} = 40plf$
$w_{S_1} = 25psf(2') = 50plf$



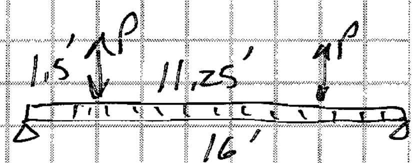
5 1/2 x 12 GLB

$w_{D_2} = 15psf(25'/2 + 1') + 12psf(9') = 138plf$
$w_{L_2} = 40plf$
$w_{S_2} = 25psf(25'/2) = 312.5plf$

$P_1 = 1210\#DL + 2015\#SL$ from Girder

$P_2 = \pm 1165\#WL \pm 750\#EL$ from BB w/ $\Omega = 3.0$

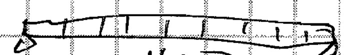
(12) $w_D = 15psf(1.33') = 20plf$
$w_L = 40psf(1.33') = 53.3plf$



$P = \pm 1900\#WL \pm 1150\#EL$ from FF w/ $\Omega = 3.0$

1 3/4 x 11 7/8 LSL

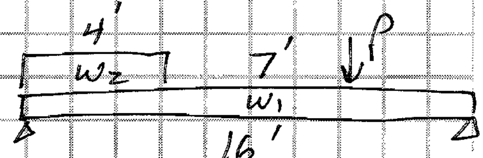
(13) $w_D = 15psf(32'/2 + 32'/2 + 16'/2) = 600plf$
$w_L = 40psf(32'/2) + 30psf(32'/2) = 1120plf$
$w_S = 25psf(16'/2) = 200plf$



4x10

4'6" span Max

(14) $w_{D_1} = 20plf$
$w_{L_1} = 53.3plf$



3 1/2 x 11 7/8 LSL

$w_{D_2} = 15psf(8'/2) = 60plf$
$w_{L_2} = 40psf(8'/2) = 160plf$

$P = 360\#DL + 960\#LL$

FOR 3402 72nd PL
JOB

DATE 11-23-20
BY MM

MYERS ENGINEERING

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

Wood Beam

DESCRIPTION: 1. Header

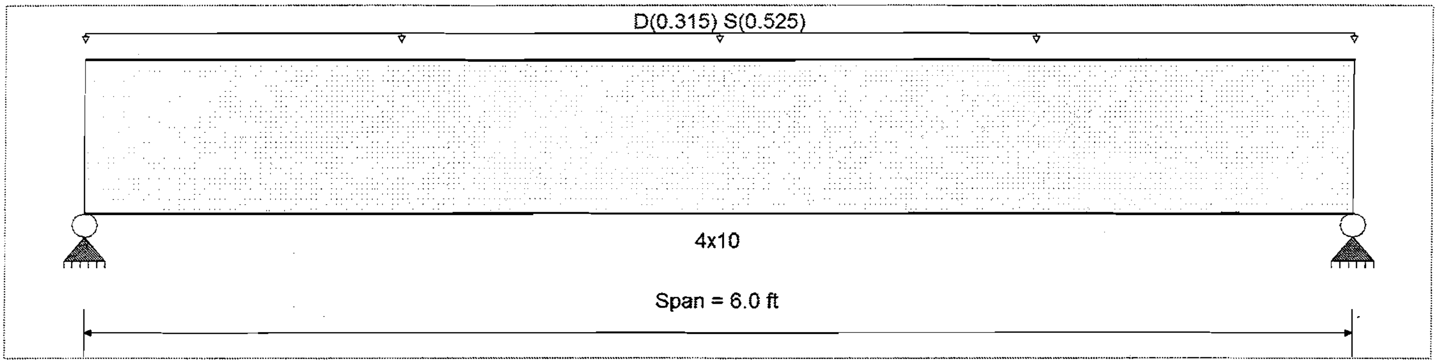
CODE REFERENCES

Calculations per NDS 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 = 0.3150, S = 0.5250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.732** 1	Maximum Shear Stress Ratio	=	**0.420** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	908.81 psi		=	86.93 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	0.042 in	Ratio =	1726 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.067 in	Ratio =	1079 >=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	2.520	2.520
Overall MINimum	1.575	1.575
D Only	0.945	0.945
+D+L	0.945	0.945
+D+S	2.520	2.520
+D+0.750L	0.945	0.945
+D+0.750L+0.750S	2.126	2.126
+0.60D	0.567	0.567
S Only	1.575	1.575

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

Wood Beam

DESCRIPTION: 1a. Header

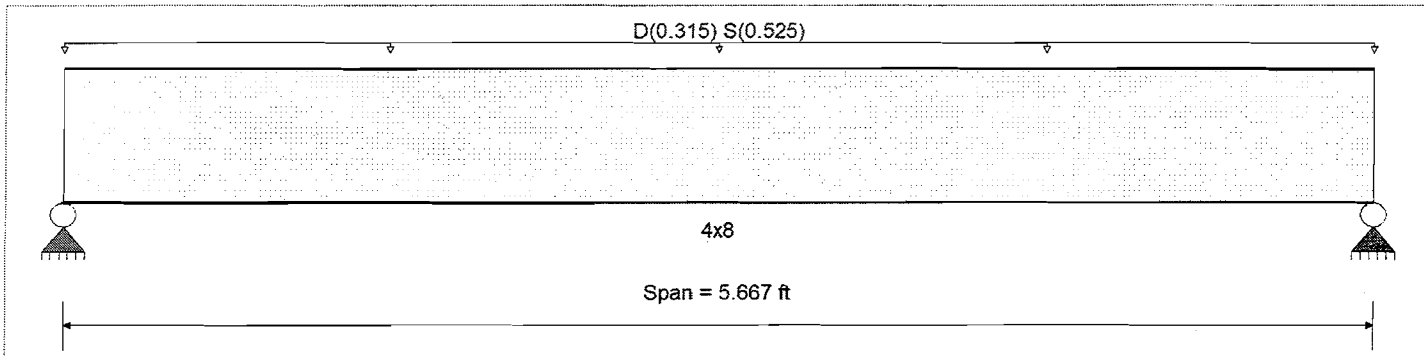
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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 = 0.3150, S = 0.5250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.981**: 1	Maximum Shear Stress Ratio	=	**0.536** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,319.73psi		=	110.92 psi
	=	1,345.50psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.834ft	Location of maximum on span	=	5.067 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.069 in	Ratio =	986 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.110 in	Ratio =	616 >=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	2.380	2.380
Overall MINimum	1.488	1.488
D Only	0.893	0.893
+D+L	0.893	0.893
+D+S	2.380	2.380
+D+0.750L	0.893	0.893
+D+0.750L+0.750S	2.008	2.008
+0.60D	0.536	0.536
S Only	1.488	1.488

2

L/480 Live Load Deflection

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

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

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

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

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

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

How to Use These Tables

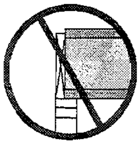
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

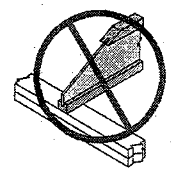
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor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

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

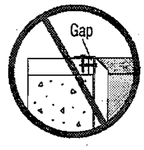
These Conditions Are <u>NOT</u> Permitted:



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



DO NOT bevel cut joist beyond inside face of wall.



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

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

Wood Beam

File: 3402 72nd PL SE_backup_1.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

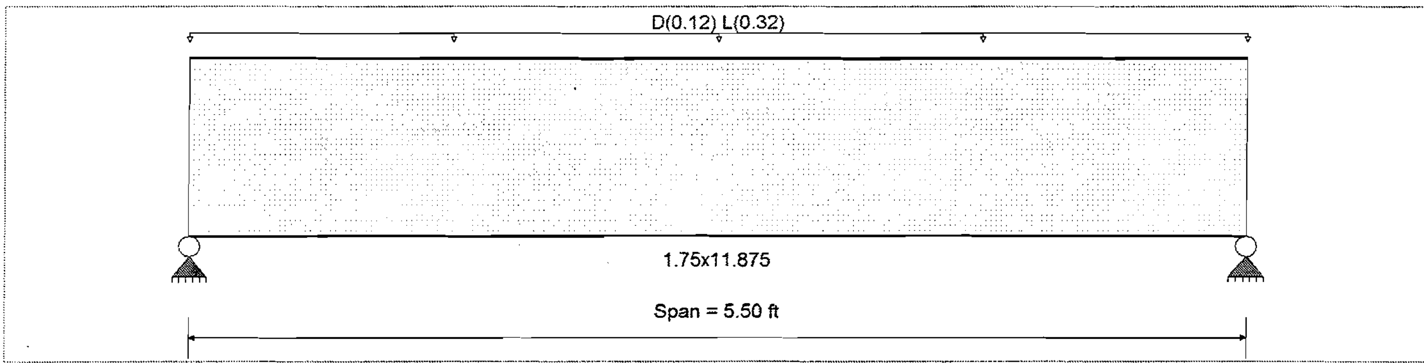
DESCRIPTION: 3. Floor beam at shower

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 = 0.120, L = 0.320 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.209 : 1	Maximum Shear Stress Ratio	=	0.181 : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	485.42 psi		=	56.10 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.750 ft	Location of maximum on span	=	4.516 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.018 in	Ratio =	3769 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.024 in	Ratio =	2741 >=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	1.210	1.210
Overall MINimum	0.880	0.880
D Only	0.330	0.330
+D+L	1.210	1.210
+D+S	0.330	0.330
+D+0.750L	0.990	0.990
+D+0.750L+0.750S	0.990	0.990
+0.60D	0.198	0.198
L Only	0.880	0.880
S Only		

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

Wood Beam

DESCRIPTION: 4. Floor beam at shower

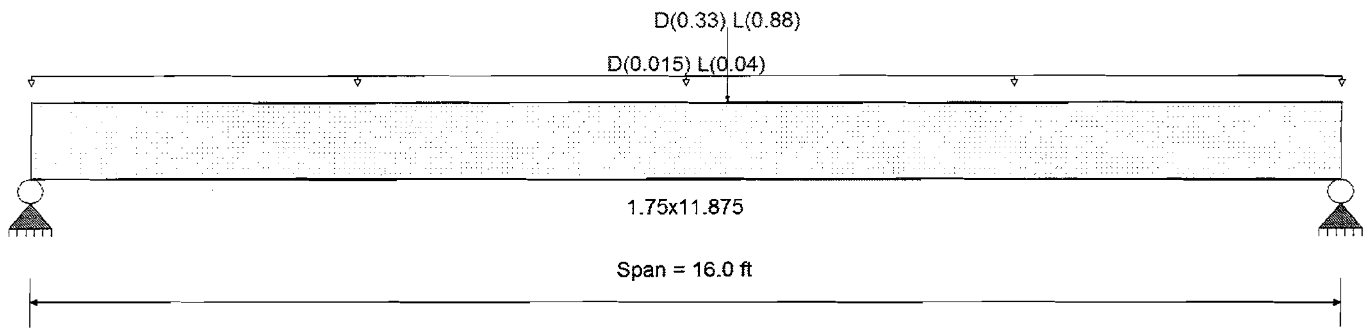
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,170.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: Trus Joist	Fc - Perp	900.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.330, L = 0.880 k @ 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.823: 1**	Maximum Shear Stress Ratio	=	**0.240 : 1**
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,913.10 psi		=	74.45 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.526 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	0.500 in	Ratio =	384 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.687 in	Ratio =	279 >=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	1.007	1.083
Overall MINimum	0.733	0.788
D Only	0.275	0.295
+D+L	1.007	1.083
+D+S	0.275	0.295
+D+0.750L	0.824	0.886
+D+0.750L+0.750S	0.824	0.886
+0.60D	0.165	0.177
L Only	0.733	0.788
S Only		

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

Wood Beam

DESCRIPTION: 5. Header supporting beam 4

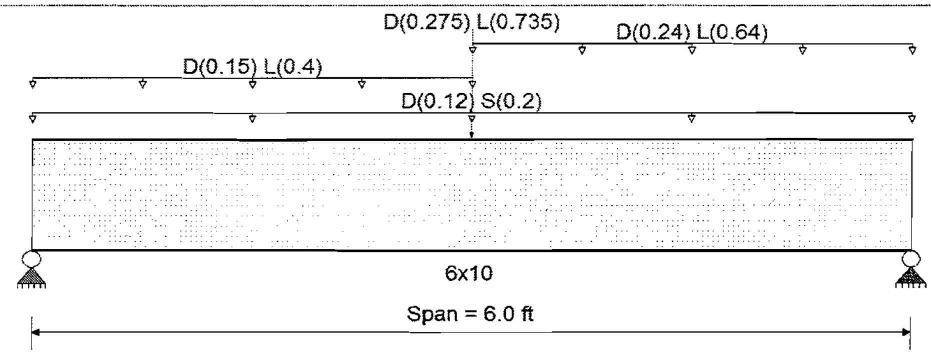
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	170 psi		
		Ft	425 psi	Density	31.21 pcf

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



Applied Loads

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

Uniform Load : D = 0.120, S = 0.20 , Tributary Width = 1.0 ft
Uniform Load : D = 0.150, L = 0.40 k/ft, Extent = 0.0 -->> 3.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.240, L = 0.640 k/ft, Extent = 3.0 -->> 6.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.2750, L = 0.7350 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.874** 1	Maximum Shear Stress Ratio	=	**0.417** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	764.78 psi		=	70.89 psi
	=	875.00 psi		=	170.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	5.212 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.041 in	Ratio =	1751 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.063 in	Ratio =	1135 >=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	2.776	3.258
Overall MINimum	0.600	0.600
D Only	1.015	1.150
+D+L	2.763	3.258
+D+S	1.615	1.750
+D+0.750L	2.326	2.731
+D+0.750L+0.750S	2.776	3.181
+0.60D	0.609	0.690
L Only	1.748	2.108
S Only	0.600	0.600

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

Wood Beam

DESCRIPTION: 6. Header

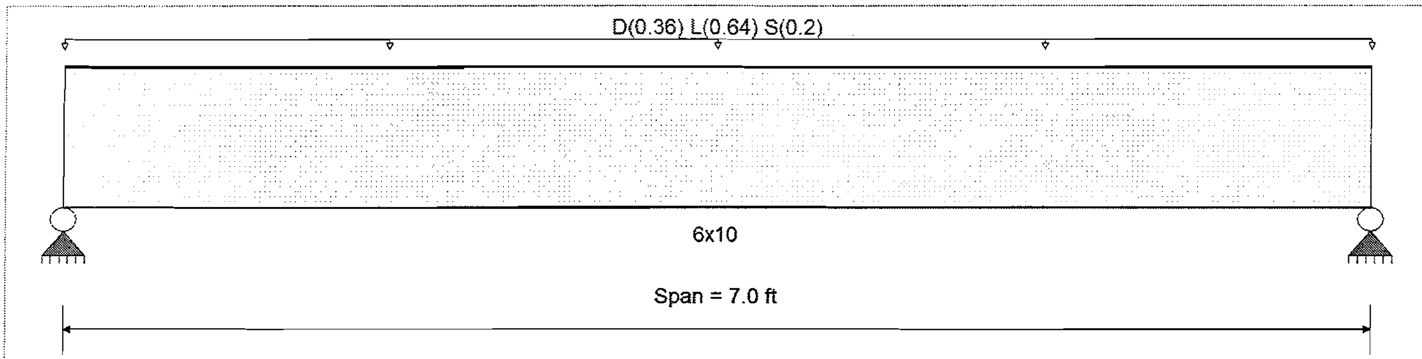
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1350 psi	Ebend- xx	1600 ksi
		Fc - Prll	925 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.1	Fv	170 psi		
		Ft	675 psi	Density	31.21 pcf

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



Applied Loads

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

Uniform Load : D = 0.360, L = 0.640, S = 0.20 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.658** 1	Maximum Shear Stress Ratio	=	**0.462** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	888.44 psi		=	78.48 psi
	=	1,350.00 psi		=	170.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.234 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.055 in	Ratio =	1518 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.086 in	Ratio =	971 >=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	3.500	3.500
Overall MINimum	0.700	0.700
D Only	1.260	1.260
+D+L	3.500	3.500
+D+S	1.960	1.960
+D+0.750L	2.940	2.940
+D+0.750L+0.750S	3.465	3.465
+0.60D	0.756	0.756
L Only	2.240	2.240
S Only	0.700	0.700

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

Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 6. Header

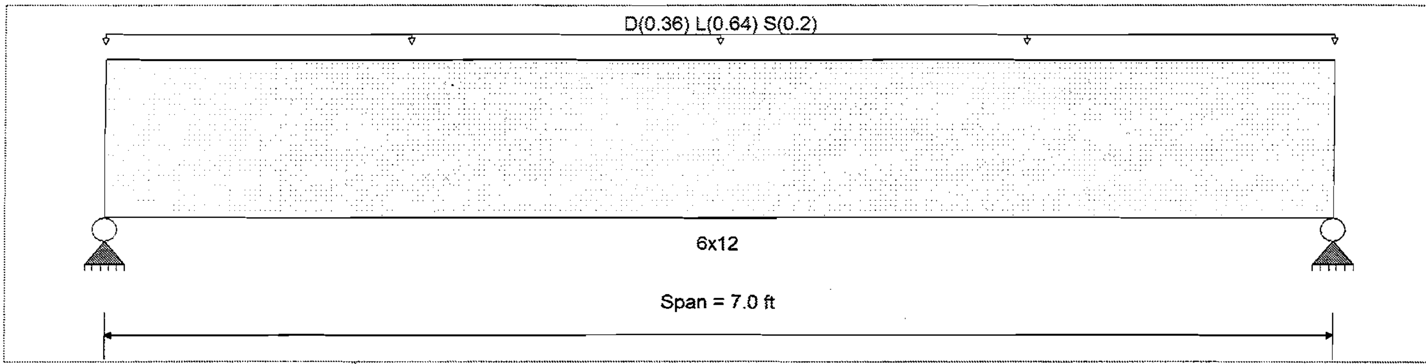
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	170 psi		
		Ft	425 psi	Density	31.21 pcf

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



Applied Loads

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

Uniform Load : D = 0.360, L = 0.640, S = 0.20 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.693**: 1	Maximum Shear Stress Ratio	=	**0.356** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	606.29psi		=	60.59 psi
	=	875.00psi		=	170.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.500ft	Location of maximum on span	=	6.055 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.038 in	Ratio =	2188 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.060 in	Ratio =	1400 >=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	3.500	3.500
Overall MINimum	0.700	0.700
D Only	1.260	1.260
+D+L	3.500	3.500
+D+S	1.960	1.960
+D+0.750L	2.940	2.940
+D+0.750L+0.750S	3.465	3.465
+0.60D	0.756	0.756
L Only	2.240	2.240
S Only	0.700	0.700

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

Wood Beam

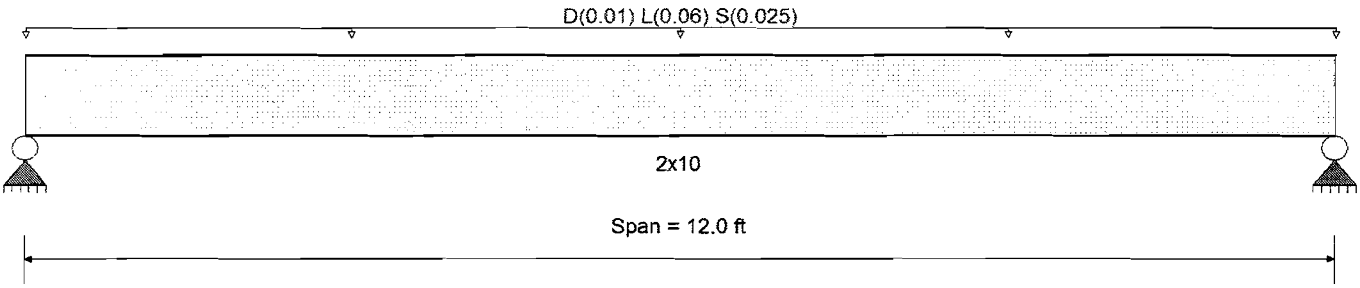
DESCRIPTION: 7. Deck Joist

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



Applied Loads

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

Uniform Load : D = 0.010, L = 0.060, S = 0.0250 ksf, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.822** 1	Maximum Shear Stress Ratio	=	**0.331** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	706.85 psi		=	39.77 psi
	=	860.20 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.000 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	0.230 in	Ratio =	624 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.283 in	Ratio =	508 >=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	0.443	0.443
Overall MINimum	0.150	0.150
D Only	0.060	0.060
+D+L	0.420	0.420
+D+S	0.210	0.210
+D+0.750L	0.330	0.330
+D+0.750L+0.750S	0.443	0.443
+0.60D	0.036	0.036
L Only	0.360	0.360
S Only	0.150	0.150

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

Wood Beam

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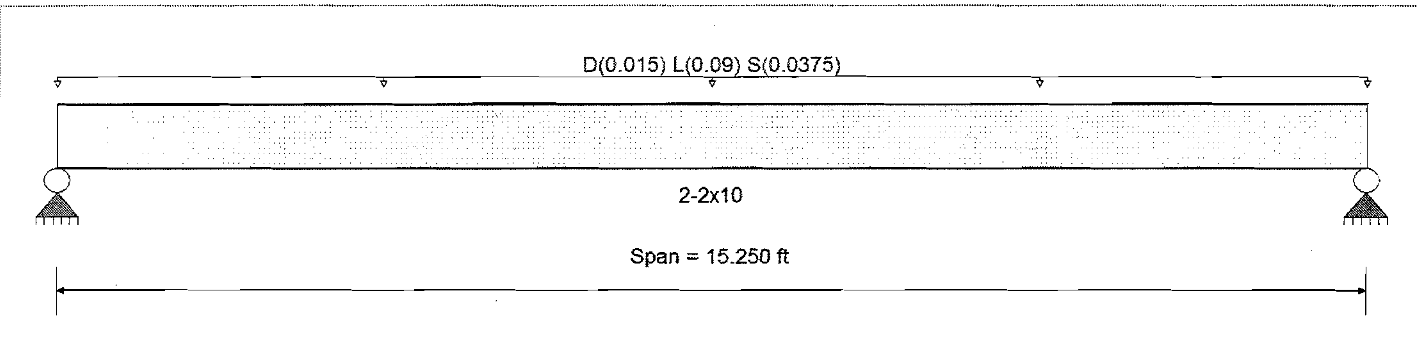
DESCRIPTION: 8. Deck Rim Beam

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



Applied Loads

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

Uniform Load : D = 0.010, L = 0.060, S = 0.0250 ksf, Tributary Width = 1.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.995** : 1	Maximum Shear Stress Ratio	=	**0.326** : 1
Section used for this span		**2-2x10**	Section used for this span		**2-2x10**
	=	856.18 psi		=	39.17 psi
	=	860.20 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.625 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	0.451 in	Ratio =	405 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.554 in	Ratio =	330 >=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	0.844	0.844
Overall MINimum	0.286	0.286
D Only	0.114	0.114
+D+L	0.801	0.801
+D+S	0.400	0.400
+D+0.750L	0.629	0.629
+D+0.750L+0.750S	0.844	0.844
+0.60D	0.069	0.069
L Only	0.686	0.686
S Only	0.286	0.286

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

Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 9. Deck Beam

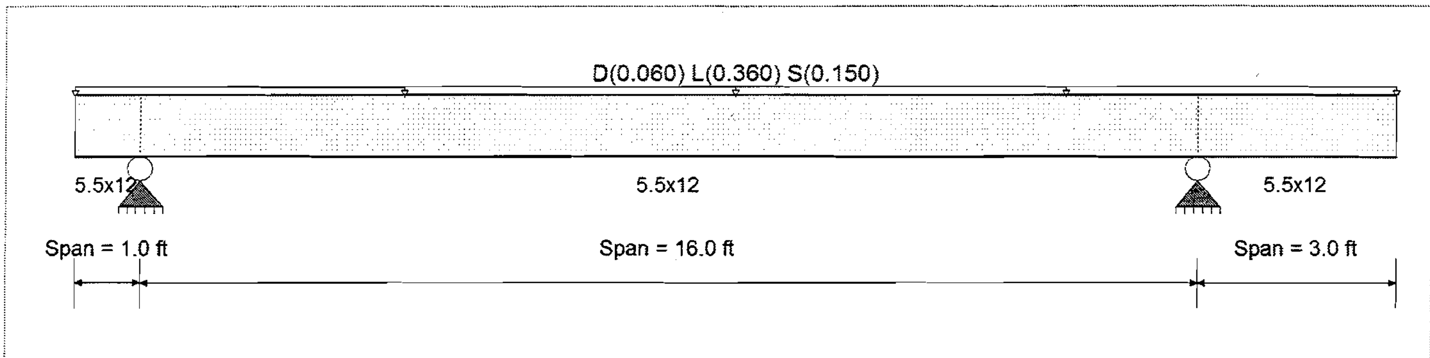
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Loads on all spans...
Uniform Load on ALL spans : D = 0.060, L = 0.360, S = 0.150 k/ft
Load for Span Number 2
Uniform Load : D = 0.0150, S = 0.0250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.488** : 1	Maximum Shear Stress Ratio	=	**0.272** : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,171.12 psi		=	72.17 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.798 ft	Location of maximum on span	=	15.059 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.342 in	Ratio =	561 >=240
Max Upward Transient Deflection	-0.068 in	Ratio =	352 >=240
Max Downward Total Deflection	0.456 in	Ratio =	421 >=240
Max Upward Total Deflection	-0.091 in	Ratio =	264 >=240

Vertical Reactions

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

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		4.142	5.248	
Overall MINimum		1.513	1.888	
D Only		0.645	0.795	
+D+L		3.795	4.845	
+D+S		2.158	2.683	
+D+0.750L		3.008	3.833	
+D+0.750L+0.750S		4.142	5.248	
+0.60D		0.387	0.477	
L Only		3.150	4.050	
S Only		1.513	1.888	

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

Wood Beam

DESCRIPTION: 10. Header in Living Room

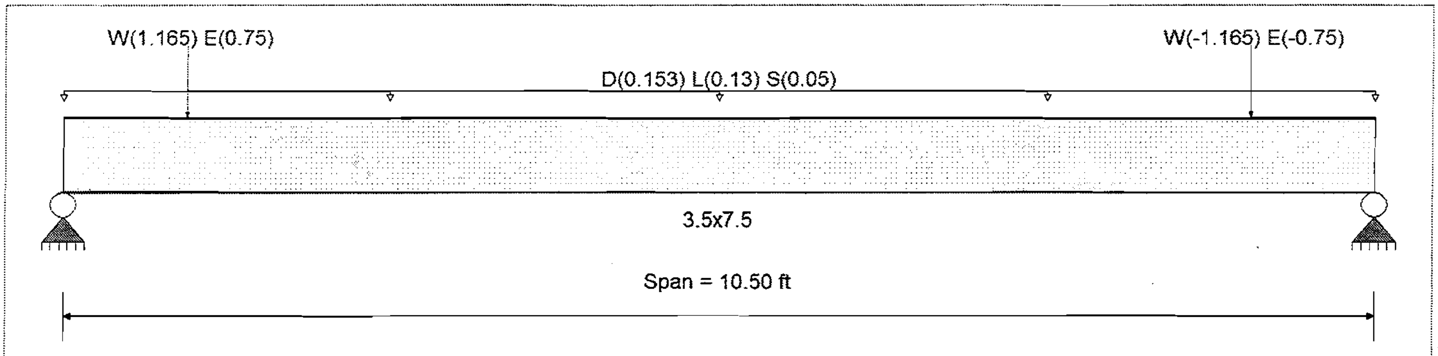
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1530, L = 0.130, S = 0.050 , Tributary Width = 1.0 ft
Point Load : W = 1.165, E = 0.750 k @ 1.0 ft
Point Load : W = -1.165, E = -0.750 k @ 9.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.594**: 1	Maximum Shear Stress Ratio	=	**0.319** : 1
Section used for this span		**3.5x7.5**	Section used for this span		**3.5x7.5**
	=	1,426.32 psi		=	135.21 psi
	=	2,400.00 psi		=	424.00 psi
Load Combination		+D+L	Load Combination		+1.105D+0.750L+0.750S+1.575E
Location of maximum on span	=	5.250 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	0.161 in	Ratio =	780 >=360
Max Upward Transient Deflection	-0.015 in	Ratio =	8166 >=360
Max Downward Total Deflection	0.358 in	Ratio =	351 >=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	1.936	1.936
Overall MINimum	-0.943	0.607
D Only	0.803	0.803
+D+L	1.486	1.486
+D+S	1.066	1.066
+D+0.750L	1.315	1.315
+D+0.750L+0.750S	1.512	1.512
+D+0.60W	1.369	0.237
+D-0.60W	0.237	1.369
+D+0.70E	1.228	0.378
+D-0.70E	0.378	1.228
+D+0.750L+0.450W	1.740	0.891
+D+0.750L-0.450W	0.891	1.740

DESCRIPTION: 10. Header in Living Room

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W	1.936	1.088
+D+0.750L+0.750S-0.450W	1.088	1.936
+D+0.750L+0.750S+0.5250E	1.831	1.193
+D+0.750L+0.750S-0.5250E	1.193	1.831
+0.60D+0.60W	1.048	-0.084
+0.60D-0.60W	-0.084	1.048
+0.60D+0.70E	0.907	0.057
+0.60D-0.70E	0.057	0.907
L Only	0.683	0.683
S Only	0.263	0.263
W Only	0.943	-0.943
-W	-0.943	0.943
E Only	0.607	-0.607
E Only * -1.0	-0.607	0.607

MYERS ENGINEERING

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

Wood Beam

DESCRIPTION: 11. Garage Door Header

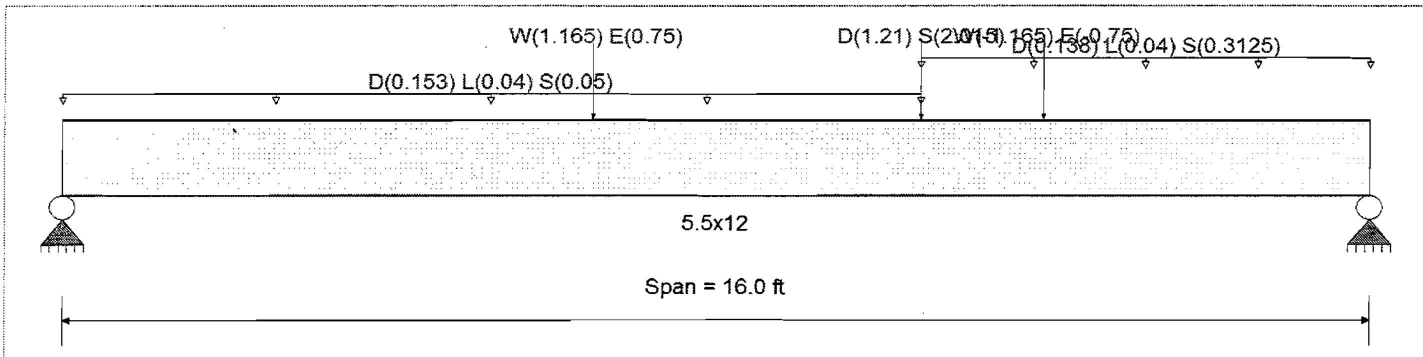
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.1530, L = 0.040, S = 0.050 k/ft, Extent = 0.0 -->> 10.50 ft, Tributary Width = 1.0 ft
- Point Load : W = 1.165, E = 0.750 k @ 6.50 ft
- Point Load : W = -1.165, E = -0.750 k @ 12.0 ft
- Uniform Load : D = 0.1380, L = 0.040, S = 0.3125 k/ft, Extent = 10.50 -->> 16.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 1.210, S = 2.015 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.657: 1	Maximum Shear Stress Ratio	=	0.330 : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,812.03psi		=	100.47 psi
	=	2,760.00psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.511 ft	Location of maximum on span	=	15.007 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.308 in	Ratio =	624 >=360
Max Upward Transient Deflection	-0.037 in	Ratio =	5182 >=360
Max Downward Total Deflection	0.572 in	Ratio =	335 >=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	3.052	4.868
Overall MINimum	-0.400	0.258
D Only	1.626	1.950
+D+L	1.946	2.270
+D+S	2.967	4.868
+D+0.750L	1.866	2.190
+D+0.750L+0.750S	2.871	4.378
+D+0.60W	1.866	1.709
+D-0.60W	1.385	2.190
+D+0.70E	1.806	1.769

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

DESCRIPTION: 11. Garage Door Header

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D-0.70E	1.445	2.130
+D+0.750L+0.450W	2.046	2.010
+D+0.750L-0.450W	1.686	2.370
+D+0.750L+0.750S+0.450W	3.052	4.198
+D+0.750L+0.750S-0.450W	2.691	4.558
+D+0.750L+0.750S+0.5250E	3.007	4.243
+D+0.750L+0.750S-0.5250E	2.736	4.514
+0.60D+0.60W	1.216	0.930
+0.60D-0.60W	0.735	1.410
+0.60D+0.70E	1.156	0.989
+0.60D-0.70E	0.795	1.350
L Only	0.320	0.320
S Only	1.341	2.918
W Only	0.400	-0.400
-W	-0.400	0.400
E Only	0.258	-0.258
E Only * -1.0	-0.258	0.258

MYERS ENGINEERING

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

Wood Beam

File: 3402 72nd PL SE_backup_1.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

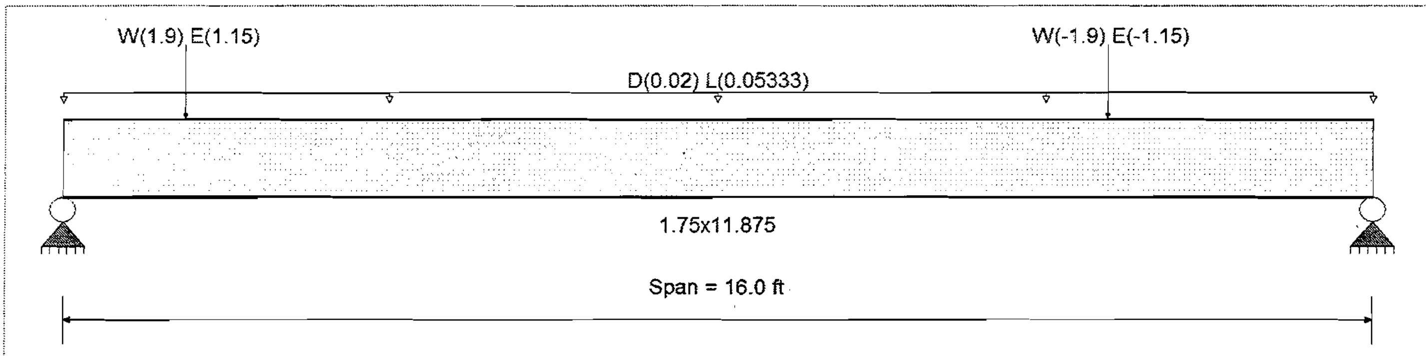
DESCRIPTION: 12. Rim Joist supporting shear wall

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,325.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	2,325.0 psi	Ebend- xx	1,550.0 ksi
		Fc - Prll	2,170.0 psi	Eminbend - xx	787.82 ksi
Wood Species	: Trus Joist	Fc - Perp	900.0 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310.0 psi		
		Ft	1,070.0 psi	Density	45.010 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.020, L = 0.05333 , Tributary Width = 1.0 ft
Point Load : W = 1.90, E = 1.150 k @ 1.50 ft
Point Load : W = -1.90, E = -1.150 k @ 12.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.469 : 1**	Maximum Shear Stress Ratio	=	**0.271 : 1**
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,744.39 psi		=	134.19 psi
	=	3,720.00 psi		=	496.00 psi
Load Combination		+1.140D-2.10E	Load Combination		+1.140D-2.10E
Location of maximum on span	=	12.730 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	0.251 in	Ratio =	765 >=360
Max Upward Transient Deflection	-0.251 in	Ratio =	765 >=360
Max Downward Total Deflection	0.339 in	Ratio =	566 >=240
Max Upward Total Deflection	-0.109 in	Ratio =	1759 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-1.336	1.336
Overall MINimum	-1.336	0.809
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
+D+0.60W	0.962	-0.642
+D-0.60W	-0.642	0.962
+D+0.70E	0.726	-0.406
+D-0.70E	-0.406	0.726
+D+0.750L+0.450W	1.081	-0.121
+D+0.750L-0.450W	-0.121	1.081

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

DESCRIPTION: 12. Rim Joist supporting shear wall

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W	1.081	-0.121
+D+0.750L+0.750S-0.450W	-0.121	1.081
+D+0.750L+0.750S+0.5250E	0.904	0.055
+D+0.750L+0.750S-0.5250E	0.055	0.904
+0.60D+0.60W	0.898	-0.706
+0.60D-0.60W	-0.706	0.898
+0.60D+0.70E	0.662	-0.470
+0.60D-0.70E	-0.470	0.662
L Only	0.427	0.427
W Only	1.336	-1.336
-W	-1.336	1.336
E Only	0.809	-0.809
E Only * -1.0	-0.809	0.809

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

Wood Beam

DESCRIPTION: 13. Beam at Crawl Space

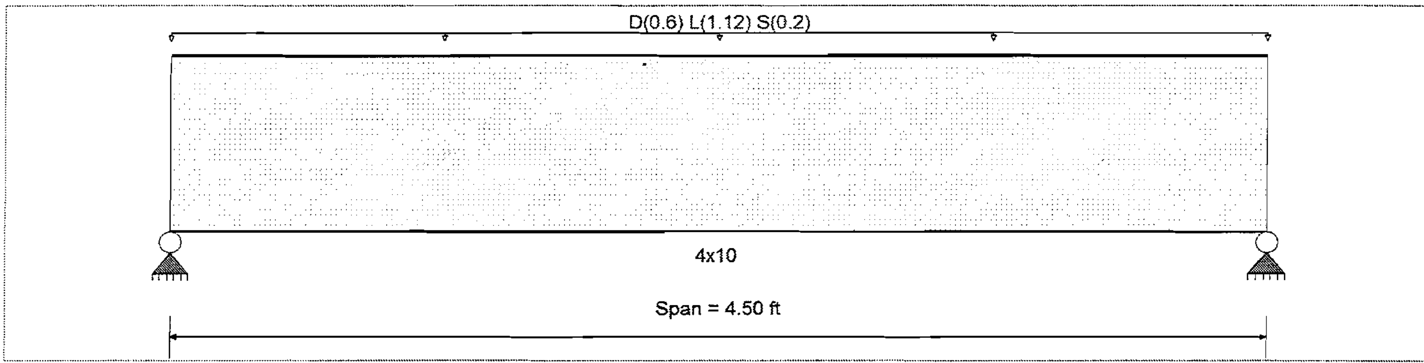
CODE REFERENCES

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 = 0.60, L = 1.120, S = 0.20 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.969 : 1**	Maximum Shear Stress Ratio	=	**0.662 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,046.75 psi		=	119.10 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.250 ft	Location of maximum on span	=	3.745 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.028 in	Ratio =	1918 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.043 in	Ratio =	1249 >=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	3.870	3.870
Overall MINimum	0.450	0.450
D Only	1.350	1.350
+D+L	3.870	3.870
+D+S	1.800	1.800
+D+0.750L	3.240	3.240
+D+0.750L+0.750S	3.578	3.578
+0.60D	0.810	0.810
L Only	2.520	2.520
S Only	0.450	0.450

FORTE WEB

MEMBER REPORT

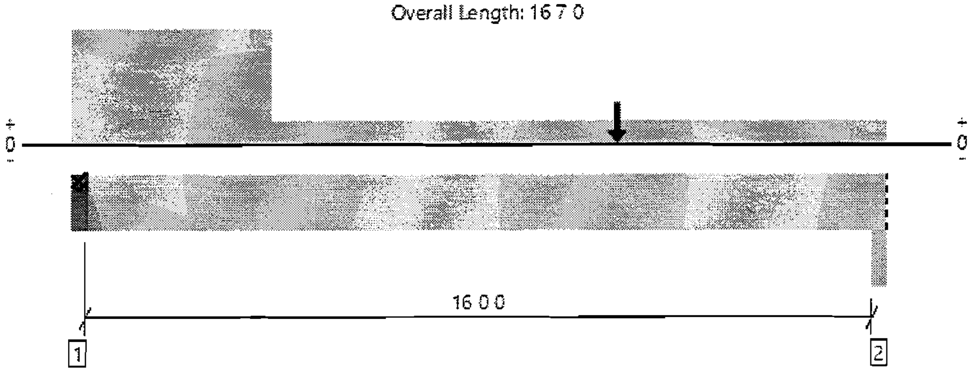
14

PASSED

Level, Floor: Joist

1 piece(s) 3 1/2" x 11 7/8" 1.55E TimberStrand® LSL @ 16" OC

Overall Length: 16 7 0



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1753 @ 0 3 8	4725 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1484 @ 15 3 10	8590	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	7340 @ 11 0 0	16591	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.290 @ 8 6 14	0.402	Passed (L/666)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.399 @ 8 6 14	0.804	Passed (L/484)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	58	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on Single 2X HF plate	3.50"	Hanger[1]	1.50"	501	1337	1838	See note [1]
2 - Beam - DF	3.50"	3.50"	1.50"	430	1148	1578	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16 4 0 o/c	
Bottom Edge (Lu)	16 4 0 o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	THA426	1.78"	4-10dx1.5	2-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 0 0 to 16 7 0	16"	15.0	40.0	Default Load
2 - Uniform (PLF)	0 0 0 to 4 0 0	N/A	60.0	160.0	
3 - Point (lb)	11 0 0	N/A	360	960	

ForteWEB Software Operator	Job Notes
Mark Myers, PE Myers Engineering LLC (253) 858-3248 myengineer@centurytel.net	

51

Weyerhaeuser

11/23/2020 11:13:51 PM UTC

ForteWEB v3.1, Engine: V8.1.5.1, Data: V8.0.1.0

File Name: 3402 72nd PL SE

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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

PROJECT : **3402 72nd Place SE**

Phone: 253-858-3248
Email: myengineer@centurytel.net

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

Maximum Load For 2-2x4 HFStud Built up Wood Post

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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

Phone: 253-858-3248
Email: myengineer@centurytel.net

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)