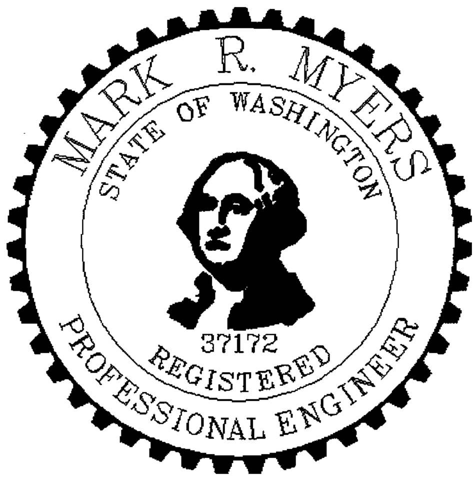
MYERS ENGINEERING

Structural Calculations



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MUST BEAR ORIGINAL BLUE INK SIGNATURE OR
DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: Chase's Corner – Lot 2
Mercer Island, WA

May 4, 2022

2018 INTERNATIONAL BUILDING CODE
100 MPH BASIC WIND, EXPOSURE B, K_{zt} = 1.03
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

3206 50th Street Court, Suite 210-B
Gig Harbor, WA 98335
Phone: 253-858-3248
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DESIGN LOADS:

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

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

WOODS : WOOD TYPE:

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

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

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

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

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

TRUSSES:

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

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

ENGINEERED I-JOISTS

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

OSHPD

Chase's Corner Lot 2

Latitude, Longitude: 47.5771, -122.2187

Date	12/1/2021, 4:45:16 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.404	MCE_R ground motion. (for 0.2 second period)
S_1	0.488	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.684	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.123	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.601	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.721	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.404	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.555	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.535	Factored deterministic acceleration value. (0.2 second)
S1RT	0.488	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.544	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.42	Factored deterministic acceleration value. (1.0 second)
PGAd	1.208	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.903	Mapped value of the risk coefficient at short periods
C_{R1}	0.897	Mapped value of the risk coefficient at a period of 1 s

LATERAL ANALYSIS :

BASED ON 2018 INTERNATIONAL BUILDING CODE (IBC)

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

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

SEISMIC DESIGN:

SEISMIC DESIGN BASED ON 2018 IBC Section 1613.1
LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT ABOVE GRADE.

Seismic Design Data:

$I_e := 1.0$ (ASCE 7-16 Table 1.5-2)

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

$S_s := 1.404$ $\quad S_1 := 0.488$ $\quad S_{ms} := 1.684$ $\quad S_{m1} := 0.878$

Equation 11.4-3 $\quad S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.12$ $\quad$ Equation 11.4-4 $\quad S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.59$

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

Roof Slope Adjustment Factor: $\quad S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{8}{12}\right)\right)} = 1.2$ $\quad S_b := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{3}{12}\right)\right)} = 1.03$

Plan Area for Each Level:

$A_1 := 1810\text{ft}^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1615\text{ft}^2$ (Upper Floor) $\quad A_{2b} := 577\text{ft}^2 \cdot S_b$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(32\text{ft}) + 2(60\text{ft})$ (Upper Floor) $\quad P_2 := 2(32\text{ft}) + 2(60\text{ft})$ (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:

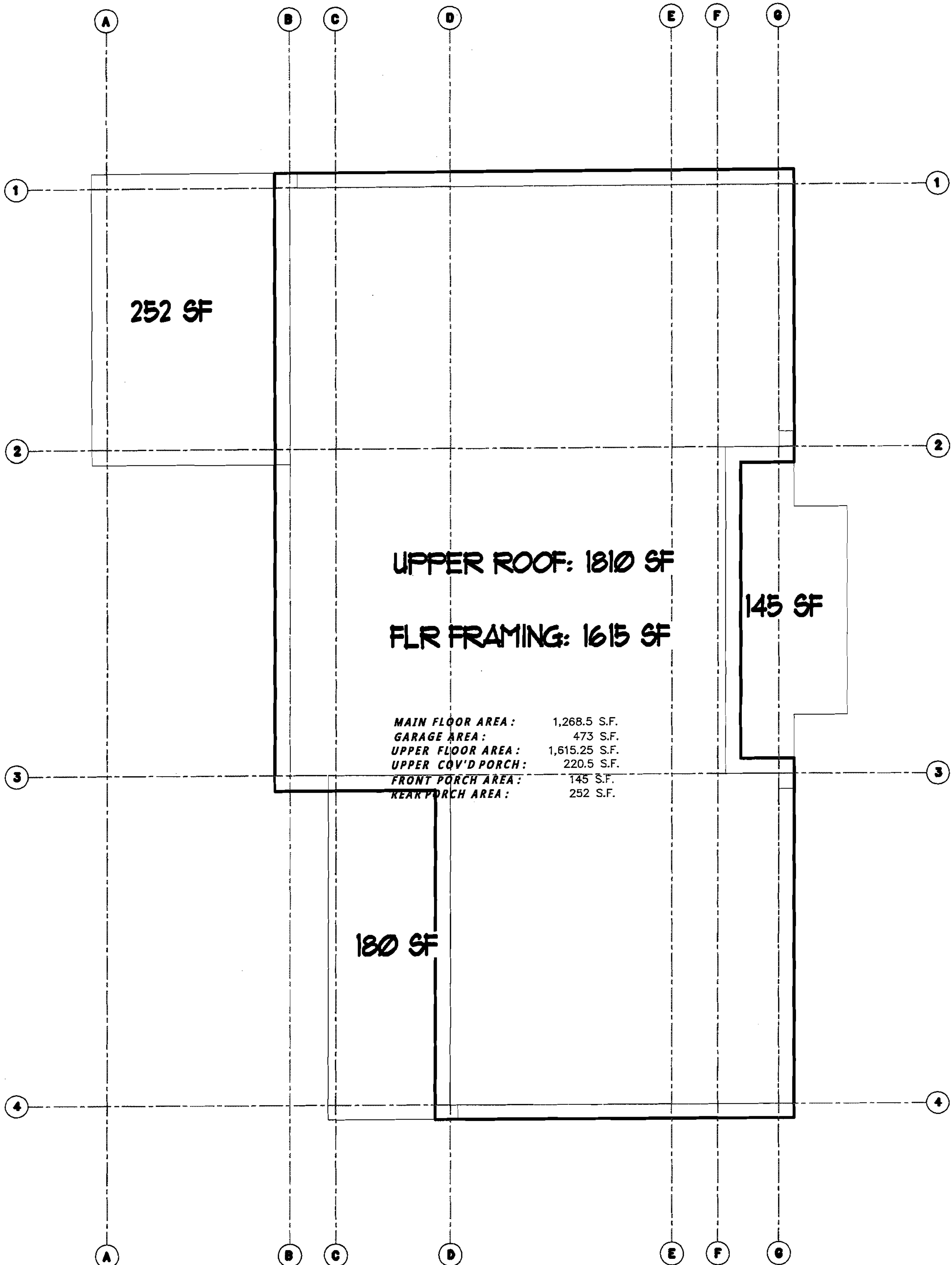
$$w_1 := 15 \cdot \text{psf} \cdot A_1 + 12 \cdot \text{psf} \cdot 4.25 \cdot \text{ft} \cdot P_1$$

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

Story Weight at Main Floor:

$$w_2 := 20 \cdot \text{psf} \cdot A_{2a} + 15\text{psf} \cdot A_{2b} + 12 \cdot \text{psf} \cdot (4.25 \cdot \text{ft} \cdot P_1 + 5\text{ft} \cdot P_2)$$

$$W := w_1 + w_2 = 103659.61 \text{ lb}$$


252 SF
UPPER ROOF: 1810 SF
FLR FRAMING: 1615 SF
145 SF
MAIN FLOOR AREA : 1,268.5 S.F.
GARAGE AREA : 473 S.F.
UPPER FLOOR AREA : 1,615.25 S.F.
UPPER COV'D PORCH : 220.5 S.F.
FRONT PORCH AREA : 145 S.F.
REAR PORCH AREA : 252 S.F.
180 SF

Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $\quad h_n := 24$ (Structural Height per ASCE 7-16 Sect. 11.2)

$T_a := C_t \cdot h_n^{\chi} = 0.22$ (ASCE 7-16 Eq. 12.8-7) $\quad T_L := 6$ (per ASCE 7-16 Fig. 22-14)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right)\cdot T_a} = 0.42$ (ASCE 7-16 Eq. 12.8-3)

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

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

Total Base Shear: $V_E := C_s \cdot W = 17903.88\,\text{lb}$

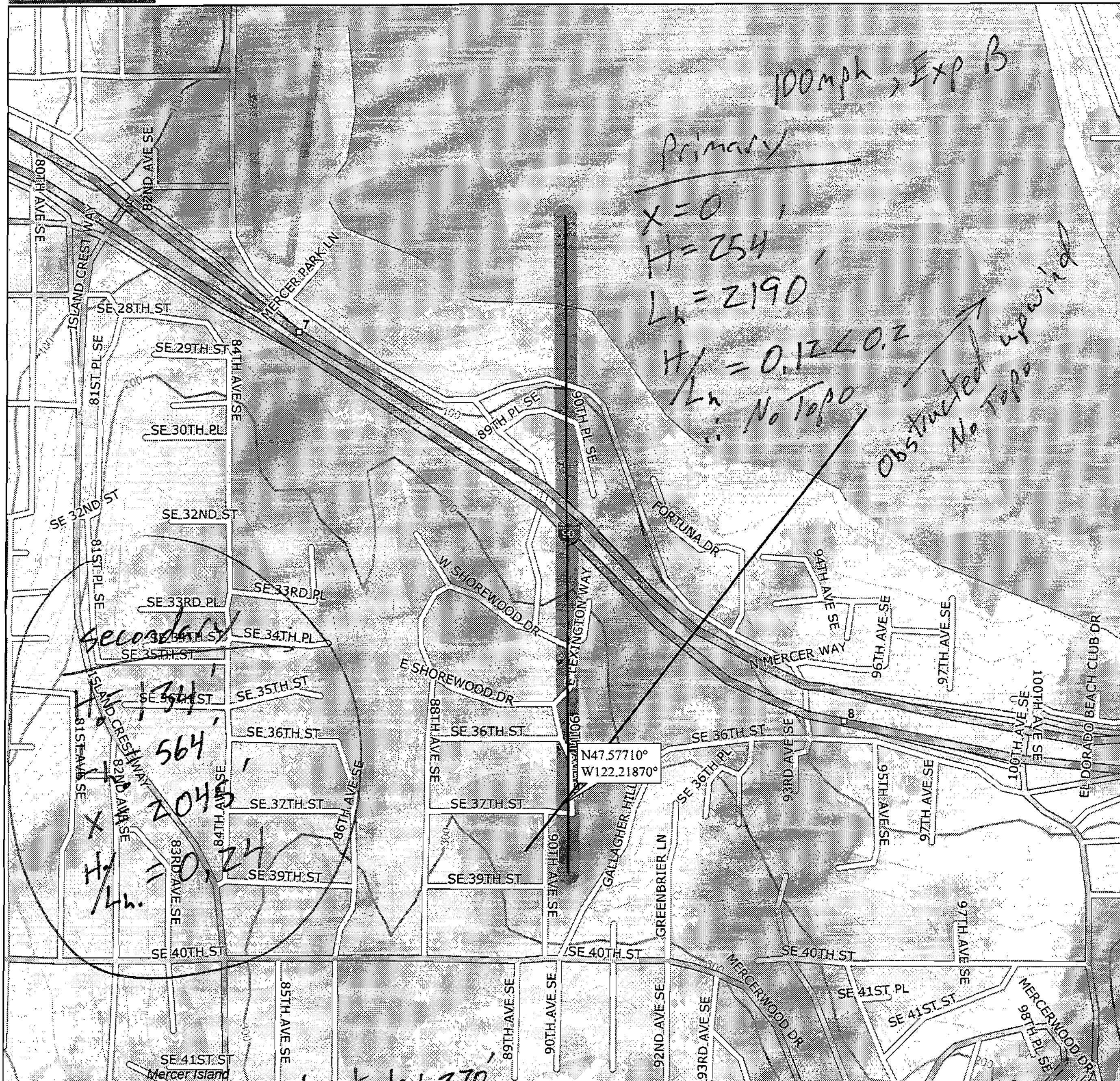
Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

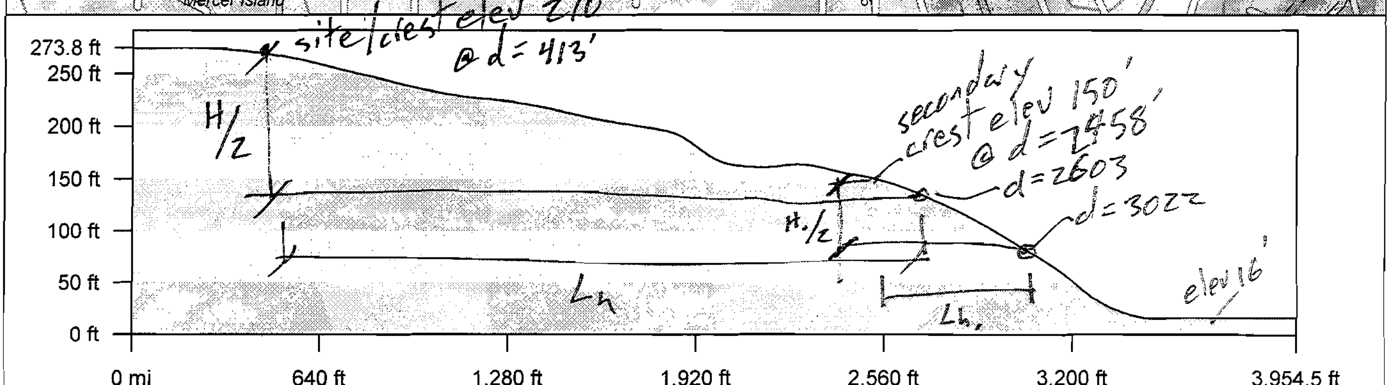
for structures having a period of 0.5 sec or less: $k := 1$

$h_1 := 19\text{ft}$ $\quad h_2 := 10\text{ft}$ (Height from base to level x)

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.56$$ $F_1 := C_{v1} \cdot V_E = 10102.42\,\text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.44$$ $F_2 := C_{v2} \cdot V_E = 7801.46\,\text{lb}$ Story Shear at Main Floor





Lin Dist: 3,938.7 ft	Terr Dist: 3,954.5 ft	Elev Gain: -257.2 ft	Avg Grade: 6
Climb Elev: 4.5 ft	Desc Elev: 261.6 ft	Max. Elev: 273.8 ft	Min. Elev: 15.3 ft
Climb Dist: 401.1 ft	Desc Dist: 3,326.2 ft		

TN
MN (14.8°E)

Scale 1 : 11,200
1" = 933.3 ft
Data Zoom 14-2

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Gig Harbor, WA 98335

PROJECT : **Chase's Corner - Lot 2**

Phone: 253-858-3248
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WIND DESIGN

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

$V := 100$ Nominal 3-Sec Gust (MPH) for Risk Category II (Figure 26.5-1B).

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

$K_e := 1$ Ground Elevation Factor (Sect. 26.9)

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

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

$x := 2045ft$ $H := 134ft$ $L_h := 564ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.75\left(\frac{H}{L_h}\right) = 0.18 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.09 \qquad K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.9 \qquad K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1.03$$

$G := 0.85$ Gust Effect Factor (ASCE 7-16 Sect. 26.11.1)

Building is an Enclosed Building as per ASCE 7-16 Sect. 26.12

$GC_{pi} := .18$ +/- Internal Pressure Coefficients (ASCE 7-16 Table 26.13-1)

Velocity Pressure Exposure Coefficient (Table 26.10-1):

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

$z_1 := 19ft$ $z_2 := 15ft$ Height from ground to level x (z_{min} = 15ft)

$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.61 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.57 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.66$$

External Pressure Coefficients w/ Roof Pitch = 3/12 (14 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 32ft$ $B_{fb} := 60ft$ $\frac{L_{fb}}{B_{fb}} = 0.53$ $\frac{h}{L_{fb}} = 0.75$

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

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

$C_{pf3} := -.55$ Leeward Roof

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

Side to Side:

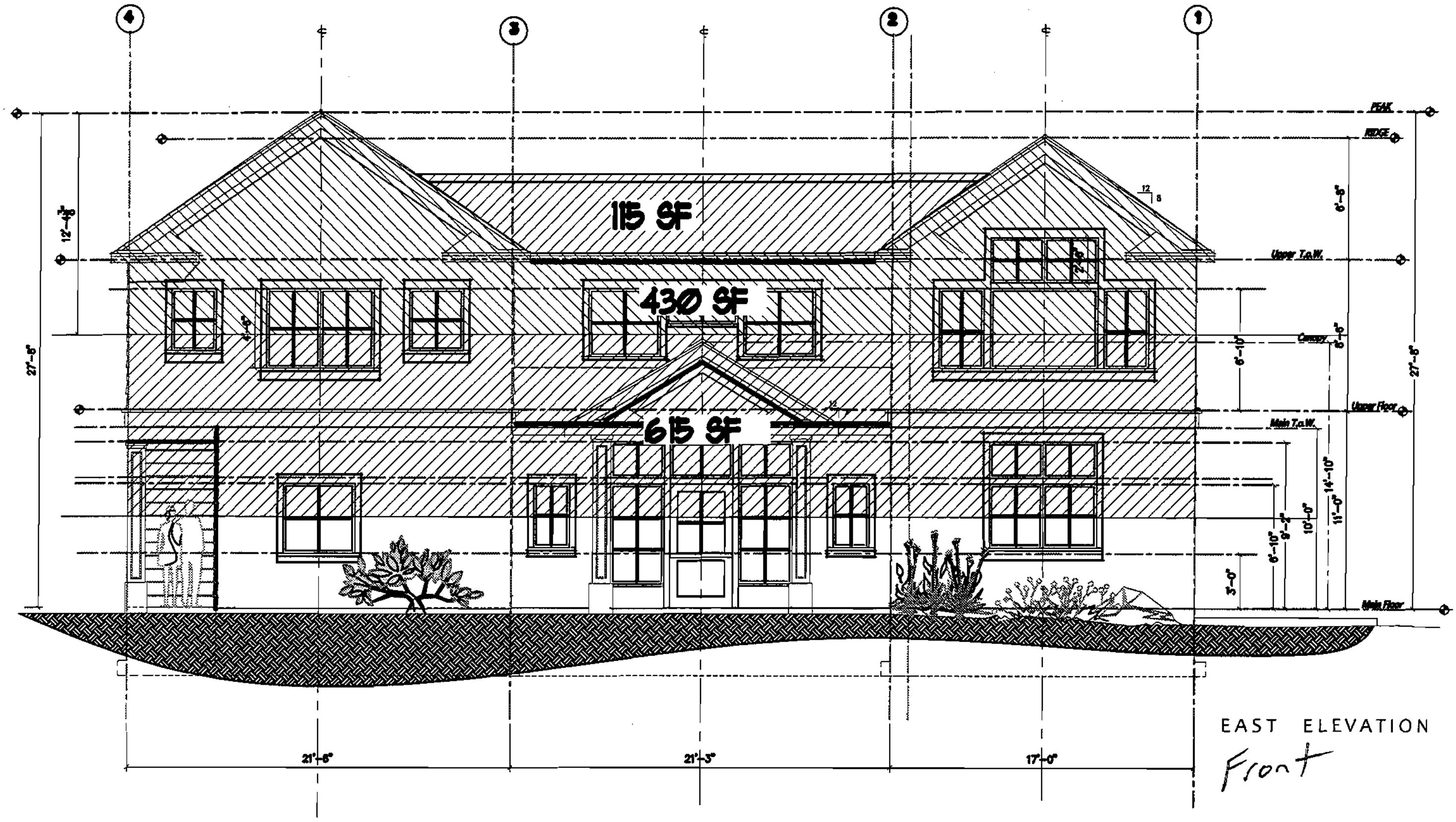
$L_{ss} := 60ft$ $B_{ss} := 32ft$ $\frac{L_{ss}}{B_{ss}} = 1.88$ $\frac{h}{L_{ss}} = 0.4$

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

$C_{ps2} := 0.34$ Windward Roof

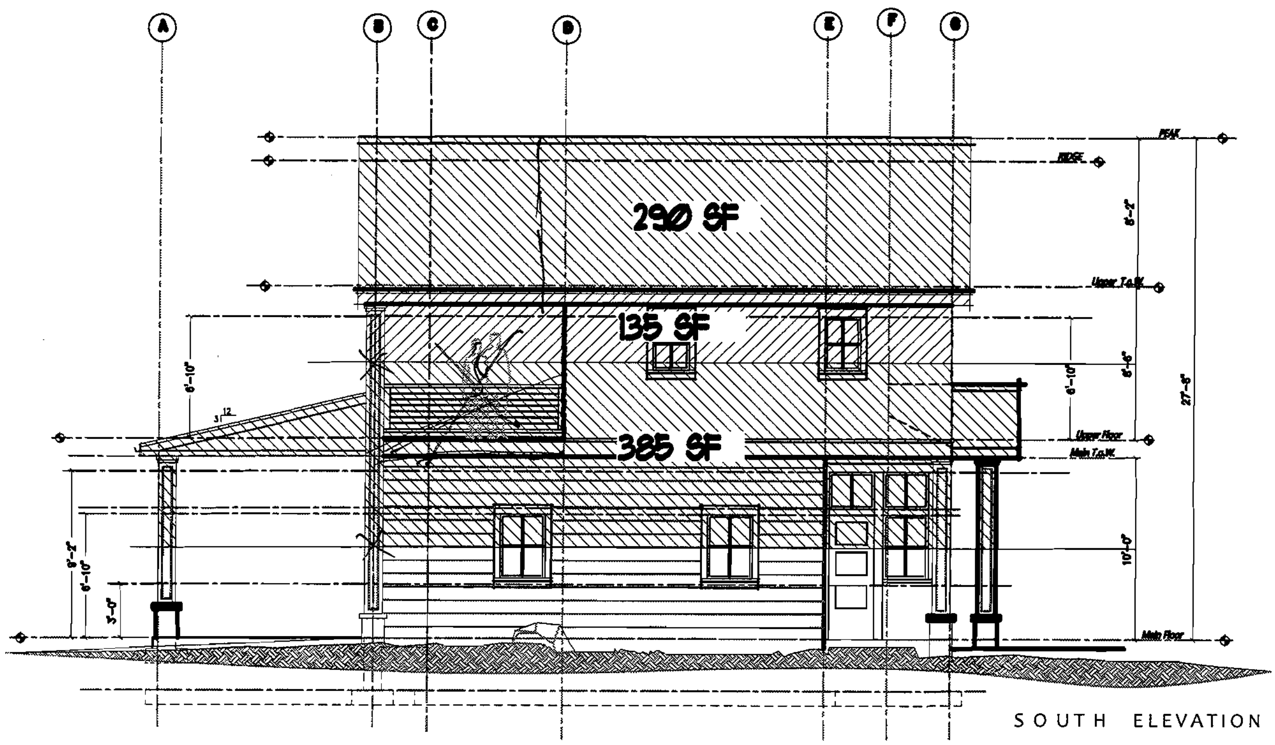
$C_{ps3} := -.6$ Leeward Roof

$C_{ps4} := -0.33$ Leeward Wall



EAST ELEVATION

Front



SOUTH ELEVATION

side

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.78$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 12.88$ $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 14.74$

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

Windward Wall Both Directions $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 9.37\,ft^{-2} \cdot lb$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 8.76\,ft^{-2} \cdot lb$

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -2.25\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -6.89\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -6.26\,ft^{-2} \cdot lb$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 4.26\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -7.51\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -4.13\,ft^{-2} \cdot lb$

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 per ASCE 7-16 Sec. 27.1-5:

$p_{wr1} - p_{lr1} = 4.63\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw1} = 15.64\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw1} = 15.02\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 11.77\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw2} = 13.51\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw2} = 12.89\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$V_{1W} := (8psf)190ft^2 + (16psf) \cdot 430 \cdot ft^2 = 8400\,lb$

Wind Pressure at Main Floor (Front to Back):

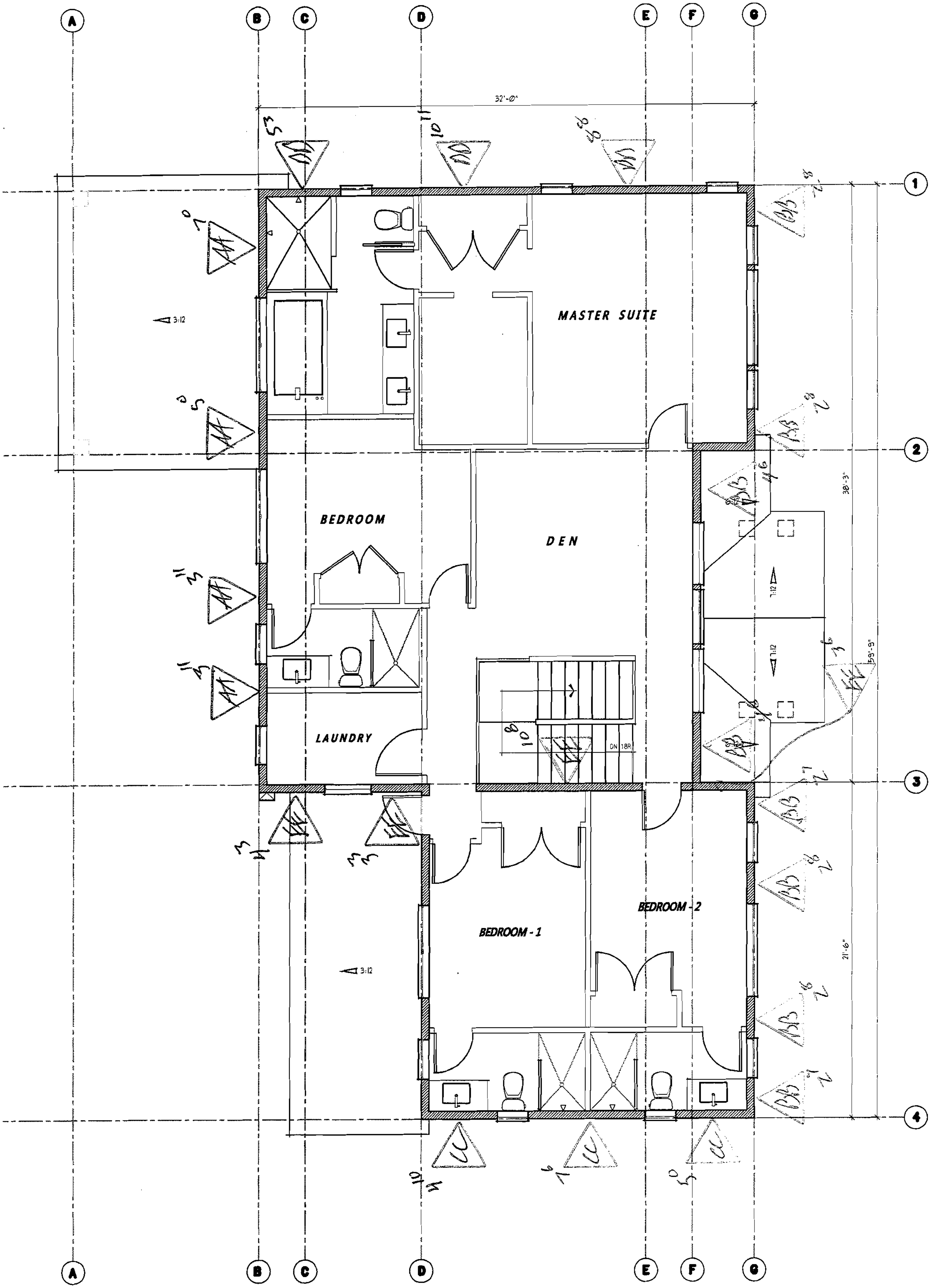
$V_{2W} := (8psf)0ft^2 + (16psf) \cdot 615 \cdot ft^2 = 9840\,lb$

Wind Pressure at Upper Roof (Side to Side):

$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 290ft^2 + (16psf) \cdot 135ft^2 = 5574.27\,lb$

Wind Pressure at Main Floor (Side to Side):

$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (16psf) \cdot 385ft^2 = 6160\,lb$



UPPER FLOOR PLAN

WALL AA:

Story Shear due to Wind: $V_{3W} = 5574.27\,lb$ Story Shear due to Seismic: $F_1 = 10102.42\,lb$

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

Shear Wall Length: $Laa := \left[2\cdot 3.92\left(\frac{7.83}{8.5}\right) + 5 + 7\right]ft = 19.22\,ft$

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

Wind Force: $vaa := \frac{\frac{0.6V_{3W}}{L_t}\cdot\frac{L_1}{2}}{Laa}$ Seismic Force: $\rho := 1.0$ $E_{aa} := \frac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Laa}$

$vaa = 87\,ft^{-1}\cdot lb$ $\frac{vaa}{C_o} = 87\,ft^{-1}\cdot lb$ $E_{aa} = 183.95\,ft^{-1}\cdot lb$ $\frac{E_{aa}}{C_o} = 183.95\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{aa} := 3.92\cdot ft$ Plate Height: $Pt := 8.5\cdot ft$

$W_{aa} := (15\cdot psf)\cdot 15\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 607.6\,lb$

Chord Force:

$CFaa_w := \frac{vaa\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 739.48\,lb$ $CFaa_s := \frac{E_{aa}\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 1563.56\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = 374.92\,lb$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 1294.49\,lb$

Simpson MSTC40

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

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

$B_p := \frac{(Z_N\cdot C_D\cdot C_o)}{vaa} = 1.88\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{aa}} = 0.89\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vaa} = 15.82\,ft$ $\frac{(Z_B\cdot C_o)}{E_{aa}} = 7.48\,ft$

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

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3206 50th Street Ct NW, Ste 210-B
Gig Harbor, WA 98335
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Email: myengineer@centurytel.net

WALL BB:

Story Shear due to Wind: $V_{3W} = 5574.27\,lb$ Story Shear due to Seismic: $F_1 = 10102.42\,lb$

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

Shear Wall Length: $Lbb := \left[2 \cdot 2.58\left(\frac{5.17}{8.5}\right) + 4 \cdot 2.67\left(\frac{5.33}{8.5}\right) + 2 \cdot 4.5\right]ft = 18.84\,ft$

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

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

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

$vbb = 88.78\,ft^{-1} \cdot lb$ $\frac{vbb}{C_o} = 88.78\,ft^{-1} \cdot lb$ $E_{bb} = 187.72\,ft^{-1} \cdot lb$ $\frac{E_{bb}}{C_o} = 187.72\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{bb} := 2.58 \cdot ft$ Plate Height: $Pt := 8.5 \cdot ft$

$W_{bb} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $DLRbb = 148.35\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_w = 754.66\,lb$ $CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_s = 1595.64\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 665.65\,lb$ $HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS}) \cdot DLRbb = 1529.95\,lb$

Simpson MSTC40 at rim or MSTC28 at flush beam

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 1.84\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.87\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vbb} = 15.5\,ft$ $\frac{(Z_B \cdot C_o)}{E_{bb}} = 7.33\,ft$

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 8400\,lb$

Story Shear due to Seismic: $F_1 = 10102.42\,lb$

Bldg Width in direction of Load: $L_t := 60\cdot ft$

Distance between shear walls: $L_1 := 21.5\cdot ft$

Shear Wall Length: $Lcc := (5 + 4.83 + 7.5)ft = 17.33\,ft$

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

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

$vcc = 52.11\,ft^{-1}\cdot lb$ $\quad \frac{vcc}{C_o} = 52.11\,ft^{-1}\cdot lb$

$E_{cc} = 73.11\,ft^{-1}\cdot lb$ $\quad \frac{E_{cc}}{C_o} = 73.11\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 4.83\cdot ft$ Plate Height: $Pt := 8.5\cdot ft$

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

$DLRcc := \frac{W_{cc}\cdot L_{cc}}{2}$ $\quad DLRcc = 603.75\,lb$

Chord Force:

$CFcc_w := \frac{vcc\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $\quad CFcc_w = 442.9\,lb$

$CFcc_s := \frac{E_{cc}\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $\quad CFcc_s = 621.44\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 80.65\,lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS})\cdot DLRcc = 354.09\,lb$

No Holdown Required

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vcc} = 3.13\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_{cc}} = 2.23\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vcc} = 26.41\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_{cc}} = 18.82\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 8400\,lb$ Story Shear due to Seismic: $F_1 = 10102.42\,lb$

Bldg Width in direction of Load: $L_t := 60 \cdot ft$ Distance between shear walls: $L_1 := 38.5 \cdot ft$

Shear Wall Length: $Ldd := (5.25 + 10.92 + 8.67)ft = 24.84\,ft$

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

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

$vdd = 65.1\,ft^{-1} \cdot lb$ $\frac{vdd}{C_o} = 65.1\,ft^{-1} \cdot lb$ $E_{dd} = 91.34\,ft^{-1} \cdot lb$ $\frac{E_{dd}}{C_o} = 91.34\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 5.25 \cdot ft$ Plate Height: $Pt := 8.5 \cdot ft$

$W_{dd} := (15 \cdot psf) \cdot 9 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRdd := \frac{W_{dd} \cdot L_{dd}}{2}$ $DLRdd = 577.5\,lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_w = 553.32\,lb$ $CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_s = 776.37\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 206.82\,lb$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 520.64\,lb$

No Holdown Required

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 2.51\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 1.79\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vdd} = 21.14\,ft$ $\frac{(Z_B \cdot C_o)}{E_{dd}} = 15.06\,ft$

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

WALL EE:

Story Shear due to Wind: $V_{1W} = 8400\,lb$

Story Shear due to Seismic: $F_1 = 10102.42\,lb$

Bldg Width in direction of Load: $L_t := 60\cdot ft$

Distance between shear walls: $L_1 := 21.5\cdot ft$ $\quad L_2 := 38.5ft$

Shear Wall Length:

$$Lee := \left[3.25\left(\frac{6.5}{8.5}\right) + 4.25 + 3.5\left(\frac{7}{8.5}\right) + 10.67\right]ft = 20.29\,ft$$

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: $vee := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lee}$

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

$vee = 124.21\,ft^{-1}\cdot lb$ $\quad \frac{vee}{C_o} = 124.21\,ft^{-1}\cdot lb$

$E_{ee} = 174.29\,ft^{-1}\cdot lb$ $\quad \frac{E_{ee}}{C_o} = 174.29\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{ee} := 3.25\cdot ft$ $\quad$ Plate Height: $Pt := 8.5\cdot ft$

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

$DLRee := \frac{W_{ee}\cdot L_{ee}}{2}$ $\quad DLRee = 406.25\,lb$

Chord Force:

$CFee_w := \frac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $\quad CFee_w = 1055.81\,lb$

$CFee_s := \frac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $\quad CFee_s = 1481.43\,lb$

Holdown Force:

$HDFee_w := CFee_w - 0.6\cdot DLRee = 812.06\,lb$

$HDFee_s := CFee_s - (0.6 - 0.14S_{DS})DLRee = 1301.53\,lb$

Simpson MSTC40

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

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

$B_n := \frac{(Z_N\cdot C_D\cdot C_o)}{vee} = 1.31\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_{ee}} = 0.94\,ft$

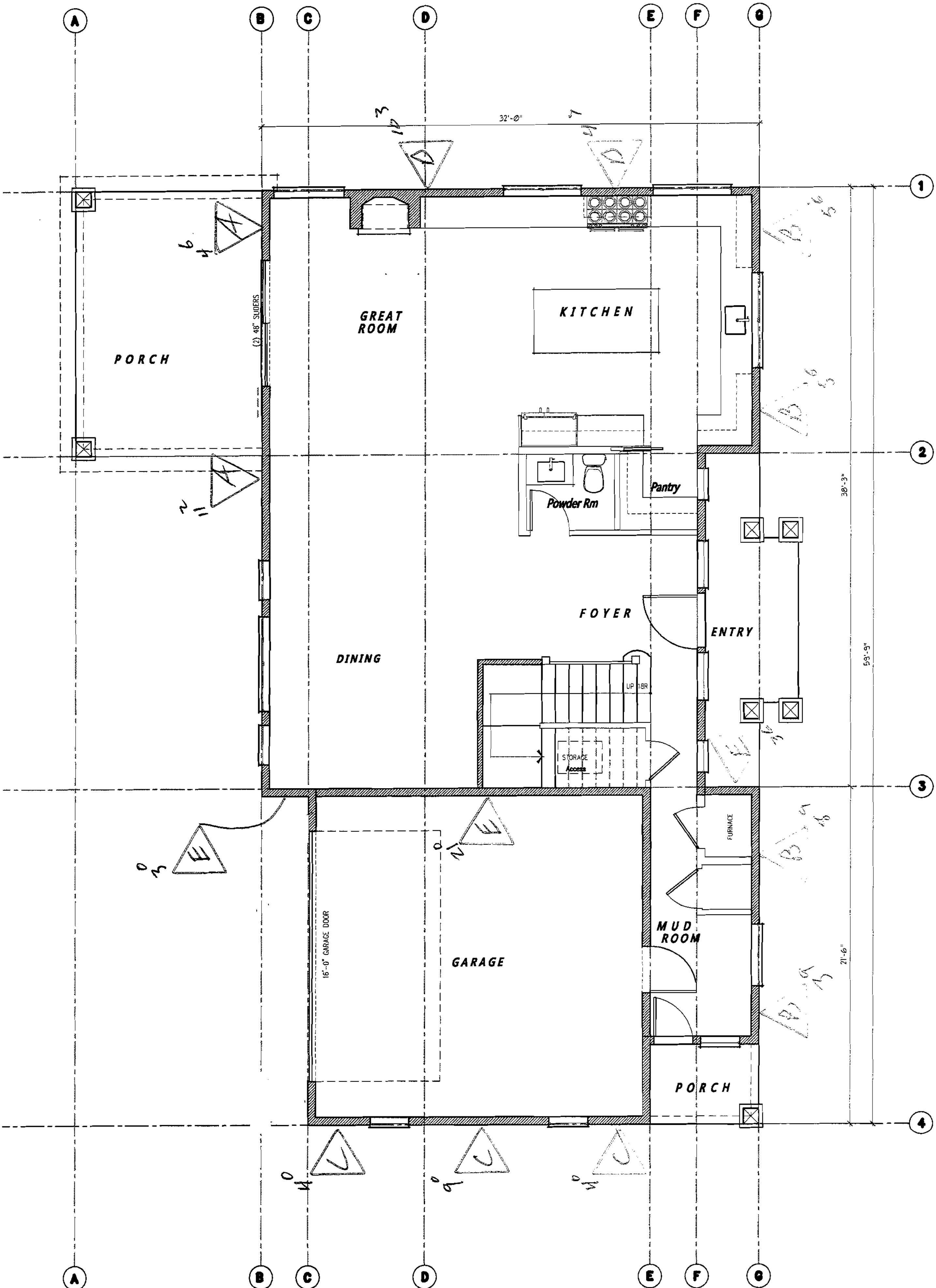
16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vee} = 11.08\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_{ee}} = 7.9\,ft$

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



MAIN FLOOR PLAN

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

PROJECT : **Chase's Corner - Lot 2**

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

WALL A:

Story Shear due to Wind: $V_{4W} = 6160\,lb$ Story Shear due to Seismic: $F_2 = 7801.46\,lb$

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

Shear Wall Length: $La := \left[11.17 + 4.5\left(\frac{9}{10}\right)\right]ft = 15.22\,ft$

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

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

$va = 231.29\,ft^{-1}\cdot lb$ $\frac{va}{C_o} = 231.29\,ft^{-1}\cdot lb$ $E_a = 411.72\,ft^{-1}\cdot lb$ $\frac{E_a}{C_o} = 411.72\,ft^{-1}\cdot lb$

P1-3: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 686 plf
Seismic Capacity = 490 plf

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

$W_a := (15\cdot psf)\cdot 6\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 450\,lb$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 2312.93\,lb$ $CFa_w + CFaa_w = 3052.42\,lb$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 4117.19\,lb$ $CFa_s + CFaa_s = 5680.74\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 2042.93\,lb$

$HDFa_w + HDFaa_w = 2417.86\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 3917.91\,lb$

$HDFa_s + HDFaa_s = 5212.41\,lb$

Simpson HDU5 w/ SB5/8x24 anchors

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.71\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 0.4\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{va} = 5.95\,ft$ $\frac{(Z_B\cdot C_o)}{E_a} = 3.34\,ft$

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

WALL B:

Story Shear due to Wind: $V_{4W} = 6160\,lb$

Story Shear due to Seismic: $F_2 = 7801.46\,lb$

Bldg Width in direction of Load: $L_t := 32\cdot ft$

Distance between shear walls: $L_1 := 32\cdot ft$

Shear Wall Length: $Lb := \left[3.75\left(\frac{7.5}{10}\right) + 8.75 + 2\cdot 5.5\right]ft = 22.56\,ft$

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

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

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

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

$vb = 156.02\,ft^{-1}\cdot lb$ $\quad \frac{vb}{C_o} = 156.02\,ft^{-1}\cdot lb$

$E_b = 277.73\,ft^{-1}\cdot lb$ $\quad \frac{E_b}{C_o} = 277.73\,ft^{-1}\cdot lb$

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

<u>Dead Load Resisting Overturning:</u> $L_b := 3.75\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_b := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$ $\quad DLRb := \frac{W_b\cdot L_b}{2}$ $\quad DLRb = 206.25\,lb$

<u>Chord Force:</u>

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $\quad CFb_w = 1560.24\,lb$

$CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $\quad CFb_s = 2777.33\,lb$

<u>Holdown Force:</u>

$HDFb_w := CFb_w - 0.6\cdot DLRb = 1436.49\,lb$

$HDFb_s := CFb_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRb = 2686\,lb$

$HDFb_w + HDFbb_w = 2102.14\,lb$

$HDFb_s + HDFbb_s = 4215.95\,lb$

Simpson HDU4 w/ SB5/8x24 Anchor

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 1.05\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.59\,ft$

16d @ 6" o.c.

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

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

$A_s := \frac{(Z_B\cdot C_o)}{vb} = 8.82\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_b} = 4.95\,ft$

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

WALL C:

Story Shear due to Wind: $V_{2W} = 9840\,\text{lb}$

Story Shear due to Seismic: $F_2 = 7801.46\,\text{lb}$

Bldg Width in direction of Load: $L_t := 60\cdot\text{ft}$

Distance between shear walls: $L_1 := 21.5\cdot\text{ft}$

Shear Wall Length: $Lc := \left[2\cdot 4\left(\frac{8}{10}\right) + 9\right]\text{ft} = 15.4\,\text{ft}$

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

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

$vc = 127.32\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{vc}{C_o} = 127.32\,\text{ft}^{-1}\cdot\text{lb}$

$E_c = 145.81\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \frac{E_c}{C_o} = 145.81\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_c := 4\cdot\text{ft}$ $\quad$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_c := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 1\text{ft}$

$DLRc := \dfrac{W_c\cdot L_c}{2}$ $\quad DLRc = 220\,\text{lb}$

Chord Force:

$CFc_w := \dfrac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_w = 1273.25\,\text{lb}$ $\quad CFc_w + CFcc_w = 1716.15\,\text{lb}$

$CFc_s := \dfrac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $\quad CFc_s = 1458.08\,\text{lb}$ $\quad CFc_s + CFcc_s = 2079.52\,\text{lb}$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 1141.25\,\text{lb}$

$HDFc_w + HDFcc_w = 1221.9\,\text{lb}$

$HDFc_s := CFc_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRc = 1360.66\,\text{lb}$

$HDFc_s + HDFcc_s = 1714.74\,\text{lb}$

Simpson LSTHD8

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

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

$B_n := \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{vc} = 1.28\,\text{ft}$ $\quad \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_c} = 1.12\,\text{ft}$

16d @ 12" o.c.

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

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

$As := \dfrac{\left(Z_B\cdot C_o\right)}{vc} = 10.81\,\text{ft}$ $\quad \dfrac{\left(Z_B\cdot C_o\right)}{E_c} = 9.44\,\text{ft}$

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

WALL D:

Story Shear due to Wind: $V_{2W} = 9840\,lb$ Story Shear due to Seismic: $F_2 = 7801.46\,lb$

Bldg Width in direction of Load: $L_t := 60 \cdot ft$ Distance between shear walls: $L_1 := 38.5 \cdot ft$

Shear Wall Length: $Ld := \left[4.58\left(\frac{9.17}{10}\right) + 10.25\right] ft = 14.45\,ft$

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

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

$vd = 242.99\,ft^{-1} \cdot lb$ $\frac{vd}{C_o} = 242.99\,ft^{-1} \cdot lb$ $E_d = 278.27\,ft^{-1} \cdot lb$ $\frac{E_d}{C_o} = 278.27\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_d := 4.58 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_d := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 8.5ft$ $DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 423.65\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 2429.92\,lb$ $CFd_w + CFdd_w = 2983.24\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 2782.66\,lb$ $CFd_s + CFdd_s = 3559.04\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 2175.73\,lb$ $HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 2595.06\,lb$

$HDFd_w + HDFdd_w = 2382.55\,lb$ $HDFd_s + HDFdd_s = 3115.7\,lb$

Simpson STHD14

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 0.67\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 0.59\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vd} = 5.66\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 4.94\,ft$

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

WALL E:

Story Shear due to Wind: $V_{2W} = 9840\,\text{lb}$ Story Shear due to Seismic: $F_2 = 7801.46\,\text{lb}$

Bldg Width in direction of Load: $L_t := 60\cdot\text{ft}$ Distance between shear walls: $L_1 := 21.5\cdot\text{ft}$ $L_2 := 38.5\text{ft}$

Shear Wall Length: $Le := \left[21 + 3 + 3.5\left(\frac{7}{10}\right)\right]\text{ft} = 26.45\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{24\cdot\text{ft}}{24.5\cdot\text{ft}}\right)\cdot 100$ $\% = 97.96$ Max Opening Height = 10ft-0in, Therefore $C_o := 0.95$ per AF&PA SDPWS Table 4.3.3.5

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

$ve = 206.88\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{ve}{C_o} = 217.77\,\text{ft}^{-1}\cdot\text{lb}$ $E_e = 236.91\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_e}{C_o} = 249.38\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_e := 3.5\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_e := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 11\text{ft}$ $DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 367.5\,\text{lb}$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 2177.69\,\text{lb}$ $CFe_w + CFee_w = 3233.51\,\text{lb}$

$CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 2493.82\,\text{lb}$ $CFe_s + CFee_s = 3975.25\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 1957.19\,\text{lb}$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 2331.09\,\text{lb}$

Simpson HDU2 w/ SSTB16 anchor

$HDFe_w + HDFee_w = 2769.26\,\text{lb}$ $HDFe_s + HDFee_s = 3632.62\,\text{lb}$

Simpson HDU4 w/ SB5/8x24 anchor

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.75\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.65\,\text{ft}$

4
16d @ ~~6~~" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{ve} = 6.32\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_e} = 5.52\,\text{ft}$

66
5/8" A.B. @ ~~60~~" o.c.

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PROJECT : **Chase's Corner - Lot 2**

Phone: 253-858-3248
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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:

$vaa \cdot \frac{Laa}{39ft} = 42.88\,ft^{-1} \cdot lb$ $\quad E_{aa} \cdot \frac{Laa}{39ft} = 90.66\,ft^{-1} \cdot lb$

Wall Lines DD:

$vdd \cdot \frac{Ldd}{32ft} = 50.53\,ft^{-1} \cdot lb$ $\quad E_{dd} \cdot \frac{Ldd}{32ft} = 70.9\,ft^{-1} \cdot lb$

Wall Lines BB:

$vbb \cdot \frac{Lbb}{59.25ft} = 28.22\,ft^{-1} \cdot lb$ $\quad E_{bb} \cdot \frac{Lbb}{59.25ft} = 59.68\,ft^{-1} \cdot lb$

Wall Lines EE:

$vee \cdot \frac{Lee}{32ft} = 78.75\,ft^{-1} \cdot lb$ $\quad E_{ee} \cdot \frac{Lee}{32ft} = 110.5\,ft^{-1} \cdot lb$

Wall Lines CC:

$vcc \cdot \frac{Lcc}{21ft} = 43\,ft^{-1} \cdot lb$ $\quad E_{cc} \cdot \frac{Lcc}{21ft} = 60.33\,ft^{-1} \cdot lb$

Wall Lines A:

$\frac{va \cdot La - vaa \cdot Laa}{39ft} = 47.38\,ft^{-1} \cdot lb$ $\quad \frac{E_a \cdot La - E_{aa} \cdot Laa}{39ft} = 70.01\,ft^{-1} \cdot lb$ $\quad \frac{va \cdot La}{39ft} = 90.26\,ft^{-1} \cdot lb$ $\quad \frac{E_a \cdot La}{39ft} = 160.68\,ft^{-1} \cdot lb$

Wall Lines B:

$\frac{vb \cdot Lb - vbb \cdot Lbb}{59.25ft} = 31.19\,ft^{-1} \cdot lb$ $\quad \frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{59.25ft} = 46.08\,ft^{-1} \cdot lb$ $\quad \frac{vb \cdot Lb}{59.25ft} = 59.41\,ft^{-1} \cdot lb$ $\quad \frac{E_b \cdot Lb}{59.25ft} = 105.76\,ft^{-1} \cdot lb$

Wall Lines C:

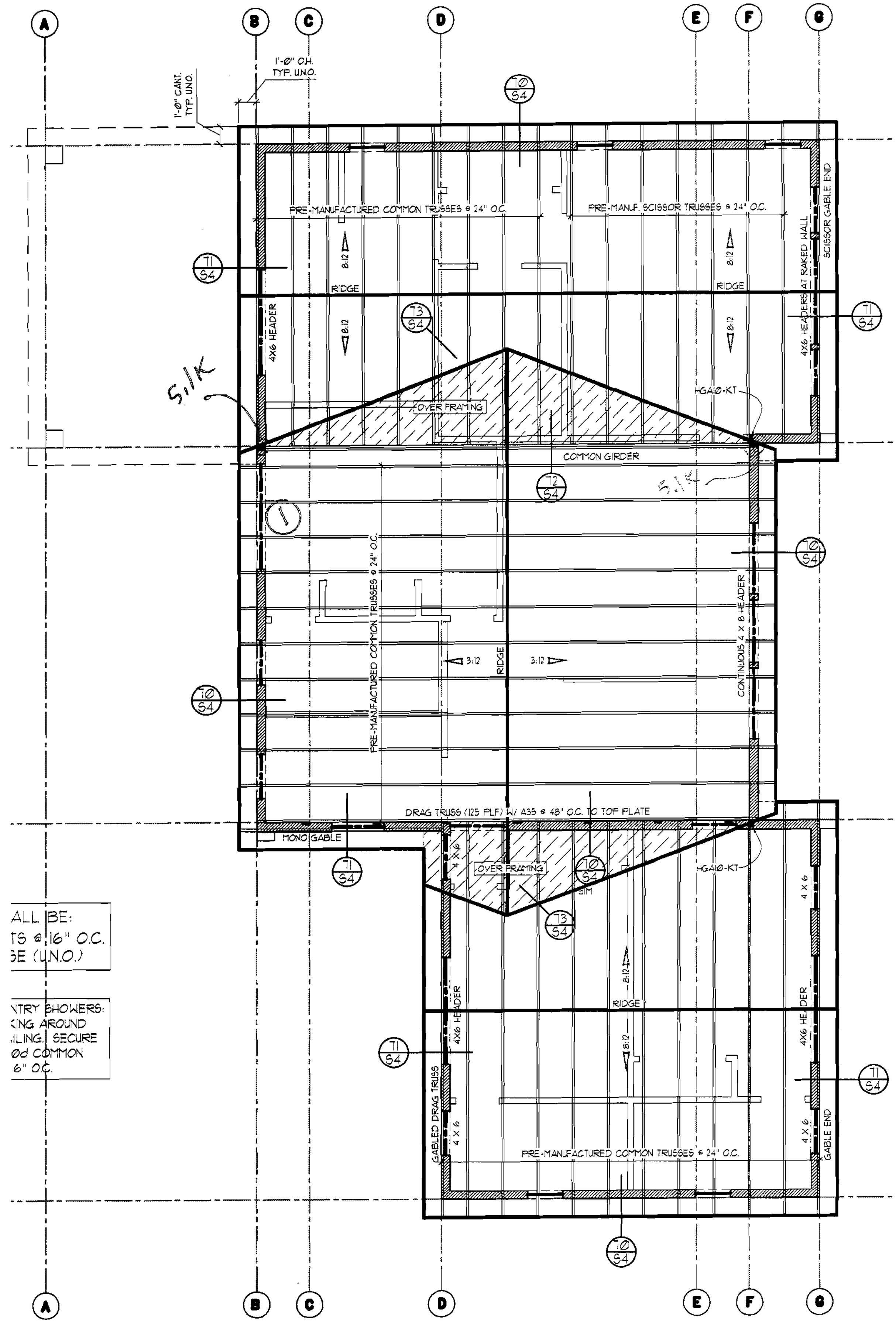
$\frac{vc \cdot Lc - vcc \cdot Lcc}{29ft} = 36.48\,ft^{-1} \cdot lb$ $\quad \frac{E_c \cdot Lc - E_{cc} \cdot Lcc}{29ft} = 33.74\,ft^{-1} \cdot lb$ $\quad \frac{vc \cdot Lc}{29ft} = 67.61\,ft^{-1} \cdot lb$ $\quad \frac{E_c \cdot Lc}{29ft} = 77.43\,ft^{-1} \cdot lb$

Wall Lines D:

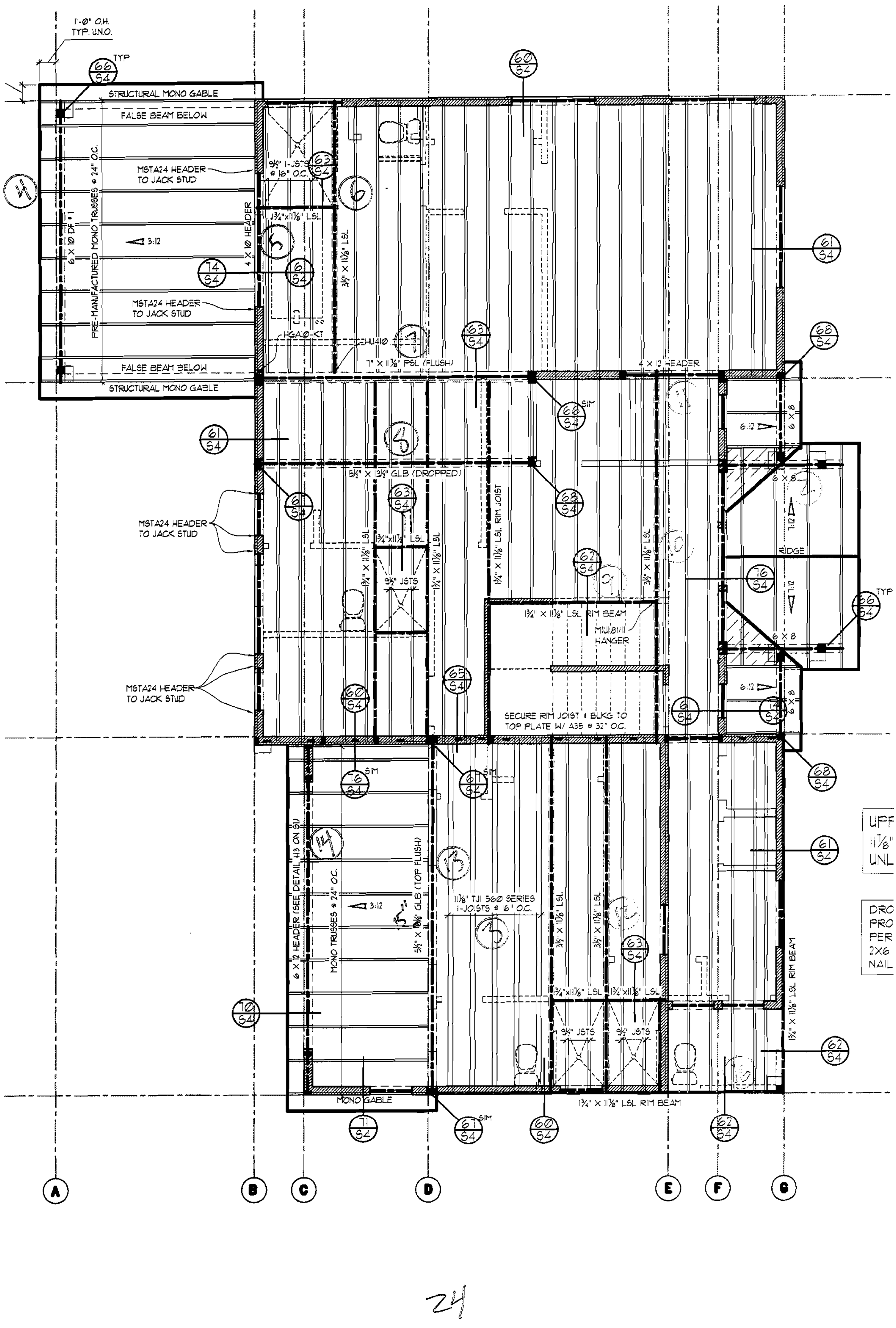
$\frac{vd \cdot Ld - vdd \cdot Ldd}{32ft} = 59.19\,ft^{-1} \cdot lb$ $\quad \frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{32ft} = 54.75\,ft^{-1} \cdot lb$ $\quad \frac{vd \cdot Ld}{32ft} = 109.72\,ft^{-1} \cdot lb$ $\quad \frac{E_d \cdot Ld}{32ft} = 125.65\,ft^{-1} \cdot lb$

Wall Line E:

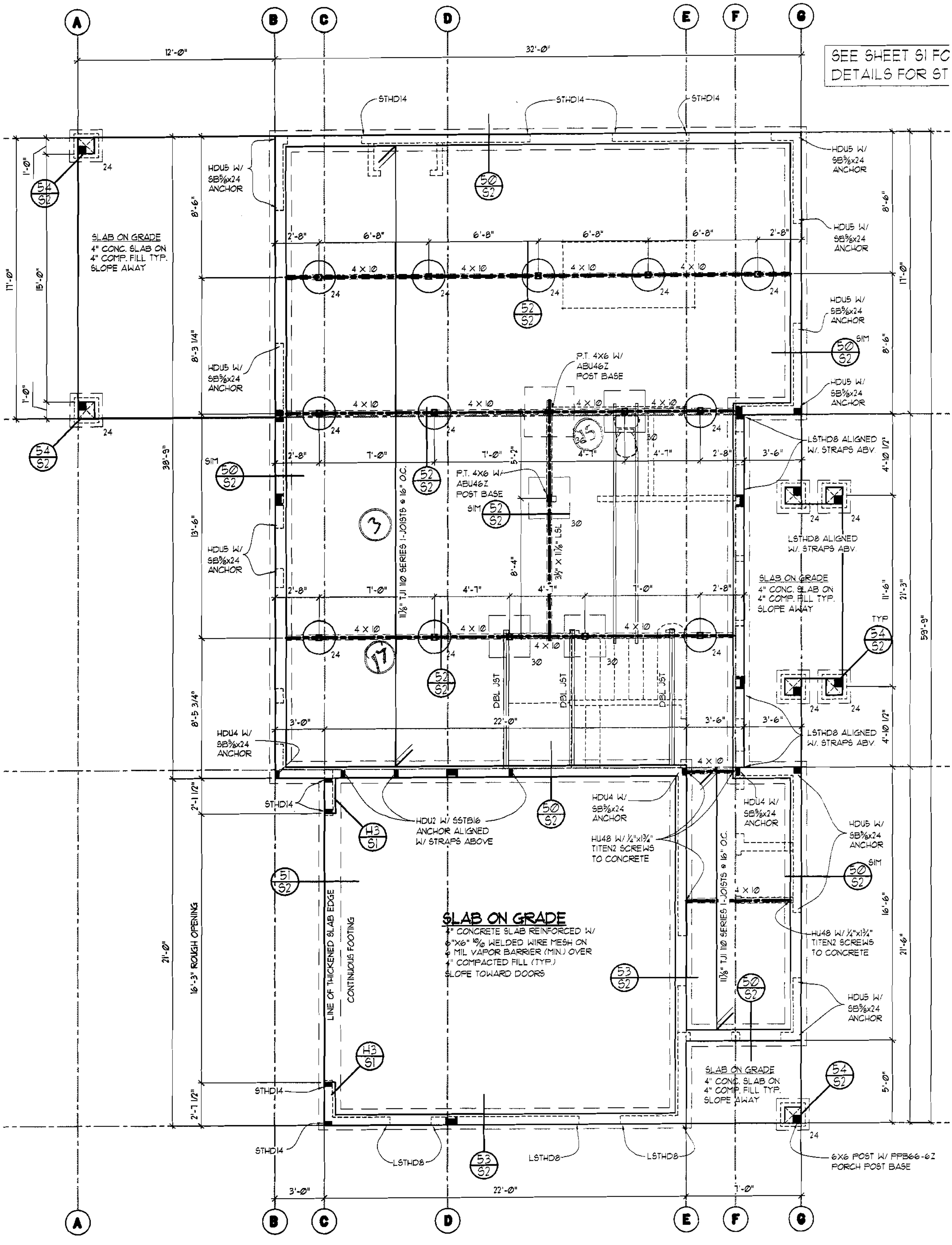
$\frac{ve \cdot Le - vee \cdot Lee}{32ft} = 92.25\,ft^{-1} \cdot lb$ $\quad \frac{E_e \cdot Le - E_{ee} \cdot Lee}{32ft} = 85.33\,ft^{-1} \cdot lb$ $\quad \frac{ve \cdot Le}{32ft} = 171\,ft^{-1} \cdot lb$ $\quad \frac{E_e \cdot Le}{32ft} = 195.82\,ft^{-1} \cdot lb$



A
B
C
D
E
F
G
1'-0" O.H. TYP. U.N.O.
1'-0" CANT. TYP. U.N.O.
10 S4
11 S4
12 S4
13 S4
PRE-MANUFACTURED COMMON TRUSSES @ 24" O.C.
PRE-MANUF. SCISSOR TRUSSES @ 24" O.C.
8:12
RIDGE
4X6 HEADER
4X6 HEADERS AT RAKED WALL
SCISSOR GABLE END
5.1K
OVER FRAMING
HGA10-KT
COMMON GIRDER
1
CONTINUOUS 4 X 8 HEADER
3:12
DRAG TRUSS (125 PLF) W/ A35 @ 48" O.C. TO TOP PLATE
MONO GABLE
SIM
4 X 6
GABLED DRAG TRUSS
GABLE END
ALL BE:
TS @ 16" O.C.
GE (U.N.O.)
NTRY SHOWERS:
KING AROUND
ILING. SECURE
0d COMMON
6" O.C.



1'-0" O.H.
TYP. U.N.O.
TYP
66/S4
STRUCTURAL MONO GABLE
FALSE BEAM BELOW
MSTA24 HEADER TO JACK STUD
PRE-MANUFACTURED MONO TRUSSES @ 24" O.C.
6 X 10 DF #1
3:12
74/S4
4 X 10 HEADER
9½" I-JSTS @ 16" O.C.
63/S4
1¾"x11⅞" LSL
61/S4
3½" X 11⅞" LSL
60/S4
61/S4
HGA10-KT
HU410
7" X 11⅞" PSL (FLUSH)
163/S4
4 X 12 HEADER
68/S4
STRUCTURAL MONO GABLE
68/S4 SIM
61/S4
6:12
6 X 8
5½" X 13½" GLB (DROPPED)
61/S4
68/S4
63/S4
1¾"x11⅞" LSL
3½" X 11⅞"
3½" X 11⅞"
1¾" X 11⅞" LSL RIM JOIST
9½" JSTS
62/S4
3½" X 11⅞" LSL
RIDGE
7:12
76/S4
TYP
66/S4
1¾" X 11⅞" LSL RIM BEAM
MIU1.81/11 HANGER
MSTA24 HEADER TO JACK STUD
60/S4
65/S4
SECURE RIM JOIST & BLKG TO TOP PLATE W/ A35 @ 32" O.C.
61/S4
74/S4
68/S4
76/S4 SIM
61/S4 SIM
6 X 12 HEADER (SEE DETAIL H3 ON S1)
MONO TRUSSES @ 24" O.C.
3:12
5½" X 11⅞" GLB (TOP FLUSH)
11⅞" TJI 360 SERIES I-JOISTS @ 16" O.C.
3½" X 11⅞"
3½" X 11⅞"
63/S4
1¾"x11⅞" LSL
1¾"x11⅞" LSL
9½" JSTS
9½" JSTS
70/S4
61/S4
62/S4
1¾" X 11⅞" LSL RIM BEAM
MONO GABLE
71/S4
67/S4 SIM
60/S4
1¾" X 11⅞" LSL RIM BEAM
62/S4
A
B
C
D
E
F
G
UPF
11⅞"
UNL
DRO
PRO
PER
2X6
NAIL
24



SEE SHEET S1 FO
DETAILS FOR ST
A
B
C
D
E
F
G
12'-0"
32'-0"
STHD14
HDU5 W/ SB⅝x24 ANCHOR
SLAB ON GRADE
4" CONC. SLAB ON 4" COMP. FILL TYP. SLOPE AWAY
4 X 10
2'-8"
6'-8"
7'-0"
4'-7"
P.T. 4X6 W/ ABU46Z POST BASE
3½" X 11⅞" LSL
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
LSTHD8 ALIGNED W/. STRAPS ABV.
HDU4 W/ SB⅝x24 ANCHOR
DBL JST
22'-0"
3'-0"
3'-6"
HDU2 W/ SSTB16 ANCHOR ALIGNED W/ STRAPS ABOVE
HU48 W/ ¼"x1¾" TITEN2 SCREWS TO CONCRETE
LINE OF THICKENED SLAB EDGE
CONTINUOUS FOOTING
16'-3" ROUGH OPENING
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
LSTHD8
6X6 POST W/ PPB66-6Z PORCH POST BASE
38'-9"
59'-9"
21'-3"
21'-0"
17'-0"
15'-0"
SIM
TYP

25

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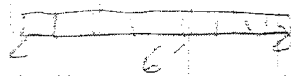
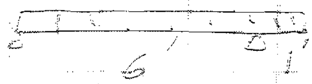
(1) $w_D = 15psf(30'/2) = 225plf$
$w_S = 25psf(30'/2) = 375plf$



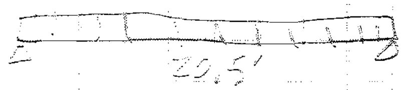
4x8 DF#2

(2) $w_D = 15psf(14'/2) = 105plf$
$w_S = 25psf(14'/2) = 175plf$



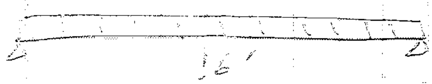
6x6 Min.

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



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

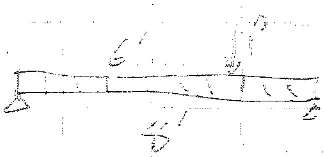
(4) $w_D = 15psf(14'/2) = 105plf$
$w_S = 25psf(14'/2) = 175plf$



6x10 DF#1

(5) $w_D = 15psf(12'/2 + 2' + 1') + 12psf(9') = 243plf$
$w_L = 40psf(1') = 40plf$
$w_S = 25psf(12'/2 + 2') = 200plf$

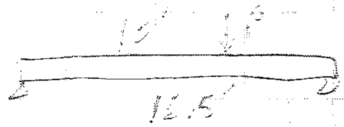
$P = 320^{\#}DL + 850^{\#}LL$



4x10

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

$P = 320^{\#}DL + 850^{\#}LL$



3½x11⅞ LSL

FOR RKK-RFA
JOB Chases Lt 2

DATE 5-2-22
BY [illegible]

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Gig Harbor, WA 98335
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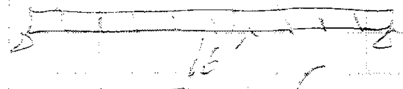
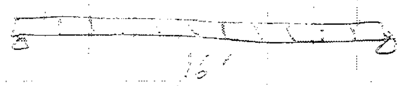
myengineer@centurytel.net

(7) $w_D = 15psf(22'/2) = 165plf$
$w_L = 40psf(22'/2) = 440plf$



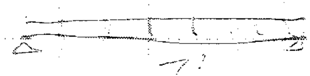
7x11 7/8 PSL

(8) $w_D = 15psf(22'/2) = 165plf$
$w_L = 40psf(22'/2) = 440plf$



5 1/8x13 1/2 GLB

(9) $w_D = 15psf(13.5'/2) = 101.3plf$
$w_L = 40psf(13.5'/2) = 270plf$

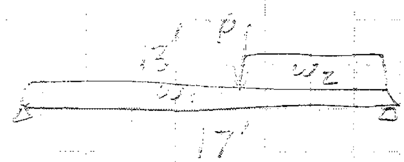


1 3/4x11 7/8 LSL

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

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

$P = 355^{\#}DL + 945^{\#}LL$ from (9)

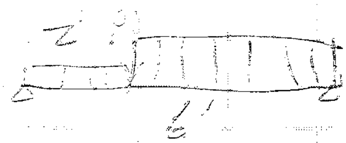


3 1/2x11 7/8 LSL

(11) $w_{D1} = 15psf(30'/2) = 225plf$
$w_{L1} = 40psf(30'/2) = 600plf$

$w_{D2} = 15psf(38'/2) = 285plf$
$w_{L2} = 40psf(38'/2) = 760plf$

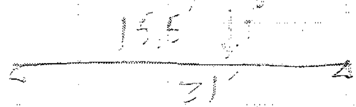
$P = 230^{\#}DL + 620^{\#}LL$ from (10)



4x12 DF#2

(12) $w_D = self$

$P = 550^{\#}DL + 1470^{\#}LL$



3 1/2x11 7/8 LSL

FOR RKK/RFA
JOB Chases Lot 2

DATE 5-2-22
BY [illegible]

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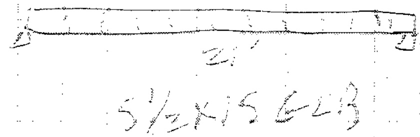
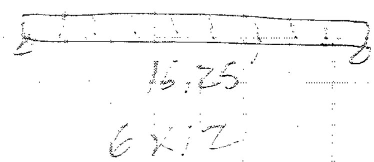
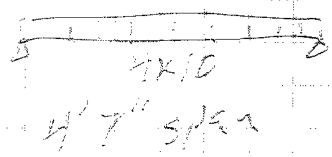
(13) $w_D = 15psf(8'/2 + 1' + 2') + 12psf(9') = 213plf$
$w_L = 40plf$
$w_S = 25psf(2' + 8'/2) = 150plf$



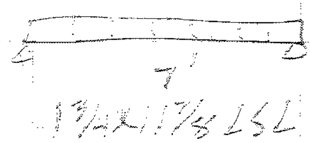
(14) $w_D = 15psf(10'/2) = 75plf$
$w_S = 25psf(10'/2) = 125plf$



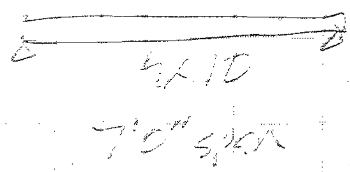
(15) $w_D = 15psf(21.5'/2 + 30'/2) = 386.3plf$
$w_L = 40psf(21.5'/2 + 30'/2) = 1030plf$



(16) $w_D = 15psf(5'/2 + 23.5'/2) + 12psf(9') = 321.8plf$
$w_L = 40psf(5'/2) = 100plf$
$w_S = 25psf(23.5'/2) = 293.8plf$



(17) $w_D = 15psf(21.5'/2) = 161.3plf$
$w_L = 40psf(21.5'/2) = 430plf$



FOR RKK/RFA
JOB Chase Lt 2

DATE 5-2-22
BY

Wood Beam

Project File: Chases Lot 2.ec6

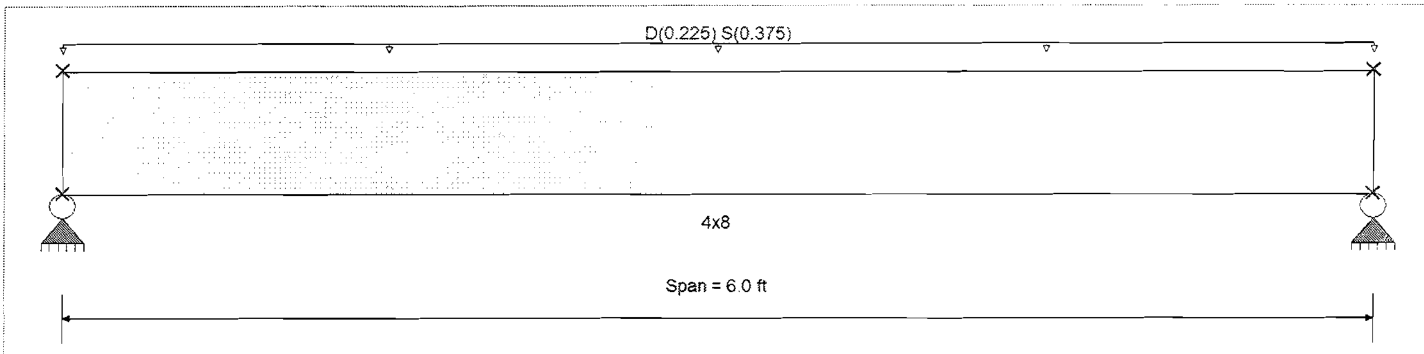
DESCRIPTION: 1. Upper Roof Header

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.2250, S = 0.3750 , Tributary Width = 1.0 ft, (Roof Trusses)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.793 : 1	Maximum Shear Stress Ratio	=	0.413 : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
fb: Actual	=	1,056.70 psi	fv: Actual	=	85.43 psi
Fb: Allowable	=	1,333.02 psi	Fv: Allowable	=	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	0.062 in	Ratio =	1164 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.099 in	Ratio =	727 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.379	0.198	0.90	1.300	1.00	1.00	1.00	1.00	0.99	1.01	396.26	1045.62	0.54	32.04	162.00
+D+L					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.341	0.178	1.00	1.300	1.00	1.00	1.00	1.00	0.99	1.01	396.26	1160.76	0.54	32.04	180.00
+D+S					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.793	0.413	1.15	1.300	1.00	1.00	1.00	1.00	0.99	2.70	1,056.70	1333.02	1.45	85.43	207.00
+D+0.750L					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.274	0.142	1.25	1.300	1.00	1.00	1.00	1.00	0.99	1.01	396.26	1447.55	0.54	32.04	225.00
+D+0.750L+0.750S					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.669	0.348	1.15	1.300	1.00	1.00	1.00	1.00	0.99	2.28	891.59	1333.02	1.22	72.08	207.00
+1.140D					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.245	0.127	1.60	1.300	1.00	1.00	1.00	1.00	0.99	1.15	451.74	1846.21	0.62	36.52	288.00
+1.105D+0.750L+0.750S					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.505	0.262	1.60	1.300	1.00	1.00	1.00	1.00	0.99	2.38	933.20	1846.21	1.28	75.45	288.00

Wood Beam

DESCRIPTION: 1. Upper Roof Header

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.129	0.067	1.60	1.300	1.00	1.00	1.00	1.00	0.99	0.61	237.76	1846.21	0.33	19.22	288.00
+0.460D					1.300	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.099	0.051	1.60	1.300	1.00	1.00	1.00	1.00	0.99	0.47	182.28	1846.21	0.25	14.74	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0990	3.022		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.800	1.800
Overall MINimum	1.125	1.125
D Only	0.675	0.675
+D+L	0.675	0.675
+D+S	1.800	1.800
+D+0.750L	0.675	0.675
+D+0.750L+0.750S	1.519	1.519
+0.60D	0.405	0.405
S Only	1.125	1.125

Project Title: Chases Corner - Lot 2
Engineer: Mark Myers, P.E.
Project ID:
Project Descr:

Wood Beam — Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.22.4.26 — MYERS ENGINEERING — (c) ENERCALC INC 1983-2022

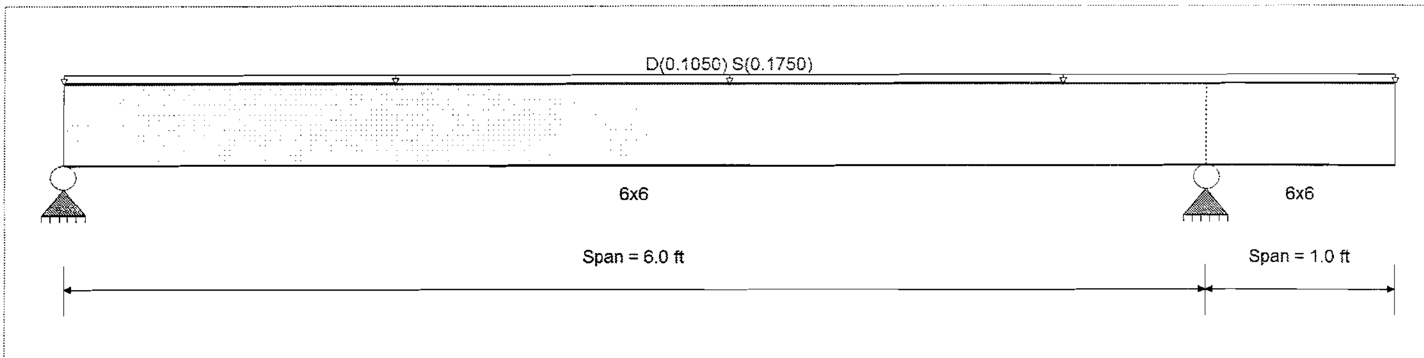
DESCRIPTION: 2. Header at Covered Porch Roof REV

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 7.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.512** 1	Maximum Shear Stress Ratio	=	**0.188** : 1
Section used for this span		**6x6**	Section used for this span		**6x6**
fb: Actual	=	515.40 psi	fv: Actual	=	36.76 psi
Fb: Allowable	=	1,006.25 psi	Fv: Allowable	=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.916 ft	Location of maximum on span	=	5.564 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.048 in	Ratio =	1484	>=360	Span: 1 : S Only
Max Upward Transient Deflection	-0.024 in	Ratio =	998	>=360	Span: 2 : S Only
Max Downward Total Deflection	0.078 in	Ratio =	928	>=240	Span: 1 : +D+S
Max Upward Total Deflection	-0.038 in	Ratio =	624	>=240	Span: 2 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.245	0.090	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.45	193.28	787.50	0.28	13.78	153.00
Length = 1.0 ft	2	0.029	0.090	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.05	22.72	787.50	0.06	13.78	153.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.512	0.188	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.19	515.40	1006.25	0.74	36.76	195.50
Length = 1.0 ft	2	0.060	0.188	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.14	60.59	1006.25	0.15	36.76	195.50
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.432	0.159	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.00	434.87	1006.25	0.63	31.02	195.50
Length = 1.0 ft	2	0.051	0.159	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.12	51.12	1006.25	0.13	31.02	195.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.157	0.058	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.51	220.33	1400.00	0.32	15.71	272.00
Length = 1.0 ft	2	0.019	0.058	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.06	25.90	1400.00	0.06	15.71	272.00

Wood Beam Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.22.4.26 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: 2. Header at Covered Porch Roof REV

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+1.105D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.325	0.119	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.05	455.16	1400.00	0.65	32.46	272.00
Length = 1.0 ft	2	0.038	0.119	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.12	53.51	1400.00	0.13	32.46	272.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.083	0.030	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.27	115.97	1400.00	0.17	8.27	272.00
Length = 1.0 ft	2	0.010	0.030	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.03	13.63	1400.00	0.03	8.27	272.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.064	0.023	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.21	88.91	1400.00	0.13	6.34	272.00
Length = 1.0 ft	2	0.007	0.023	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	10.45	1400.00	0.03	6.34	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0776	2.983		0.0000	0.000
	2	0.0000	2.983	+D+S	-0.0384	1.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.817	1.143	
Overall MINimum	0.510	0.715	
D Only	0.306	0.429	
+D+S	0.817	1.143	
+D+0.750S	0.689	0.965	
+0.60D	0.184	0.257	
S Only	0.510	0.715	

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

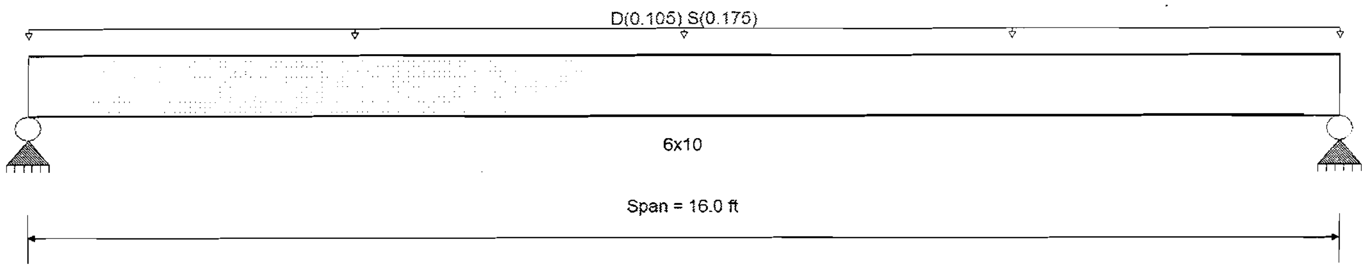
DESCRIPTION: 4. Main Floor Cov'd Porch Beam

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.1050, S = 0.1750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.837: 1	Maximum Shear Stress Ratio	=	0.298 : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb: Actual	=	1,299.66 psi	fv: Actual	=	58.20 psi
Fb: Allowable	=	1,552.50 psi	Fv: Allowable	=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.241 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.413 in	Ratio =	465 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.661 in	Ratio =	290 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.401	0.143	0.90	1.000	1.00	1.00	1.00	1.00	1.00	3.36	487.37	1215.00	0.76	21.83	153.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.361	0.128	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.36	487.37	1350.00	0.76	21.83	170.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.837	0.298	1.15	1.000	1.00	1.00	1.00	1.00	1.00	8.96	1,299.66	1552.50	2.03	58.20	195.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.289	0.103	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.36	487.37	1687.50	0.76	21.83	212.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.706	0.251	1.15	1.000	1.00	1.00	1.00	1.00	1.00	7.56	1,096.59	1552.50	1.71	49.11	195.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.257	0.091	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.83	555.61	2160.00	0.87	24.88	272.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.531	0.189	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.91	1,147.76	2160.00	1.79	51.40	272.00

Wood Beam

Project File: Chases Lot 2.ec6

DESCRIPTION: 4. Main Floor Cov'd Porch Beam

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.135	0.048	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.02	292.42	2160.00	0.46	13.10	272.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.104	0.037	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.55	224.19	2160.00	0.35	10.04	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6605	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.240	2.240
Overall MINimum	1.400	1.400
D Only	0.840	0.840
+D+L	0.840	0.840
+D+S	2.240	2.240
+D+0.750L	0.840	0.840
+D+0.750L+0.750S	1.890	1.890
+0.60D	0.504	0.504
S Only	1.400	1.400

Project Title:	Chases Corner - Lot 2
Engineer:	Mark Myers, P.E.
Project ID:	
Project Descr:	

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 5. Header at Great Rm Slider

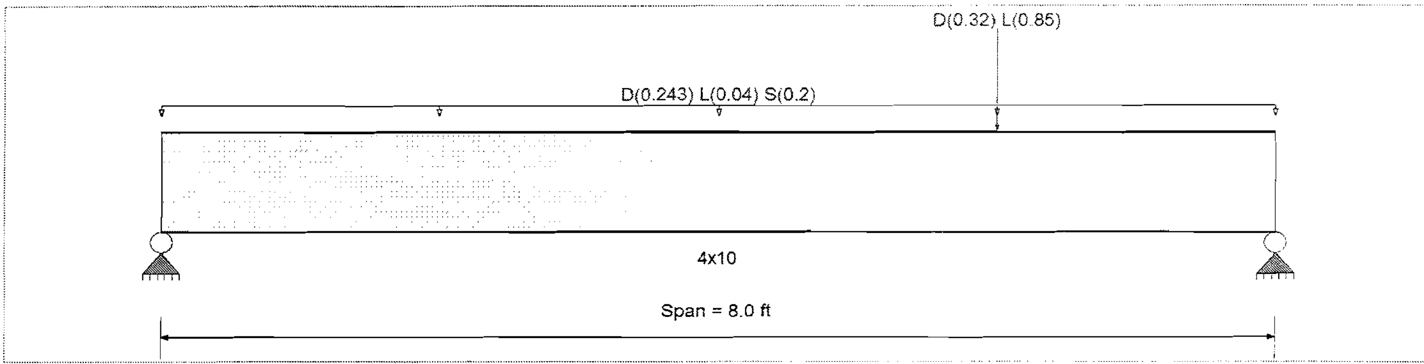
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.2430, L = 0.040, S = 0.20 , Tributary Width = 1.0 ft
Point Load : D = 0.320, L = 0.850 k @ 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.854** 1	Maximum Shear Stress Ratio	=	**0.468** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
fb: Actual	=	1,060.09psi	fv: Actual	=	96.79 psi
Fb: Allowable	=	1,242.00psi	Fv: Allowable	=	207.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	4.555ft	Location of maximum on span	=	7.241 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.050 in	Ratio =	1912 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.139 in	Ratio =	689 >=240	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.563	0.294	0.90	1.200	1.00	1.00	1.00	1.00	1.00	2.28	547.49	972.00	1.03	47.61	162.00
+D+L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.798	0.462	1.00	1.200	1.00	1.00	1.00	1.00	1.00	3.59	861.96	1080.00	1.79	83.15	180.00
+D+S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.749	0.375	1.15	1.200	1.00	1.00	1.00	1.00	1.00	3.87	930.74	1242.00	1.68	77.64	207.00
+D+0.750L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.578	0.330	1.25	1.200	1.00	1.00	1.00	1.00	1.00	3.25	780.53	1350.00	1.60	74.26	225.00
+D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.854	0.468	1.15	1.200	1.00	1.00	1.00	1.00	1.00	4.41	1,060.09	1242.00	2.09	96.79	207.00
+1.140D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.361	0.188	1.60	1.200	1.00	1.00	1.00	1.00	1.00	2.60	624.14	1728.00	1.17	54.27	288.00
+1.105D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

DESCRIPTION: 5. Header at Great Rm Slider

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 8.0 ft	1	0.647	0.353	1.60	1.200	1.00	1.00	1.00	1.00	1.00	4.65	1,117.42	1728.00	2.20	101.79	288.00
+0.60D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.190	0.099	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.37	328.49	1728.00	0.62	28.56	288.00
+0.460D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.146	0.076	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.05	251.84	1728.00	0.47	21.90	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.1393	4.117		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.931	2.410
Overall MINimum	0.800	0.800
D Only	1.052	1.212
+D+L	1.425	2.010
+D+S	1.852	2.012
+D+0.750L	1.331	1.810
+D+0.750L+0.750S	1.931	2.410
+0.60D	0.631	0.727
L Only	0.373	0.798
S Only	0.800	0.800

Project Title: Chases Corner - Lot 2
Engineer: Mark Myers, P.E.
Project ID:
Project Descr:

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 6. Floor beam at shower

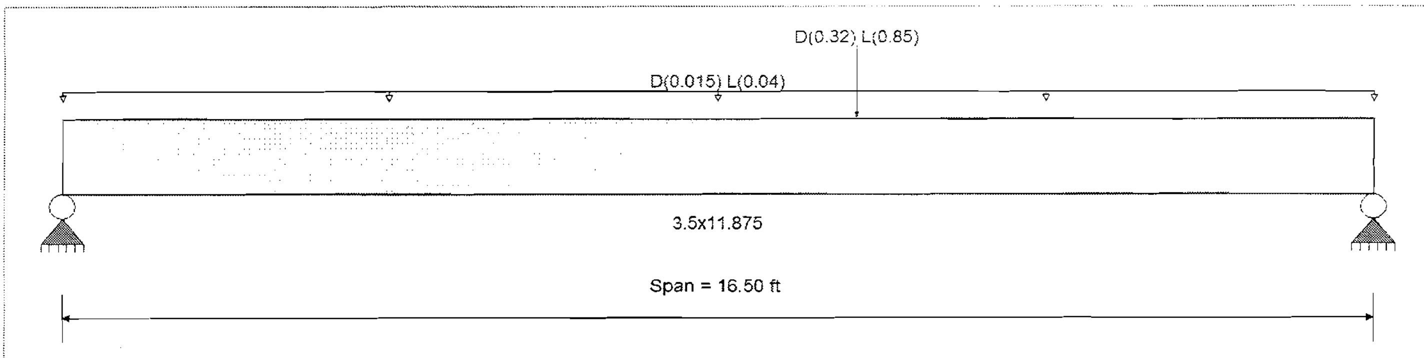
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.320, L = 0.850 k @ 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.401**: 1	Maximum Shear Stress Ratio	=	**0.129** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb: Actual	=	932.94 psi	fv: Actual	=	40.05 psi
Fb: Allowable	=	2,325.00 psi	Fv: Allowable	=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.996 ft	Location of maximum on span	=	15.536 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.260 in	Ratio =	760 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.358 in	Ratio =	552 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.122	0.039	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.75	254.96	2092.50	0.30	10.94	279.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.401	0.129	1.00	1.000	1.00	1.00	1.00	1.00	1.00	6.40	932.94	2325.00	1.11	40.05	310.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.095	0.031	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.75	254.96	2673.75	0.30	10.94	356.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.263	0.085	1.25	1.000	1.00	1.00	1.00	1.00	1.00	5.23	763.45	2906.25	0.91	32.78	387.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.286	0.092	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.23	763.45	2673.75	0.91	32.78	356.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	**1**	0.078	0.025	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.99	290.66	3720.00	0.35	12.48	496.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

DESCRIPTION: 6. Floor beam at shower

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 16.50 ft	1	0.212	0.068	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.42	790.22	3720.00	0.94	33.93	496.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.041	0.013	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.05	152.98	3720.00	0.18	6.57	496.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.032	0.010	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.80	117.28	3720.00	0.14	5.03	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.3581	8.611		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.915	1.163
Overall MINimum	0.665	0.845
D Only	0.250	0.318
+D+L	0.915	1.163
+D+S	0.250	0.318
+D+0.750L	0.748	0.952
+D+0.750L+0.750S	0.748	0.952
+0.60D	0.150	0.191
L Only	0.665	0.845
S Only		

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

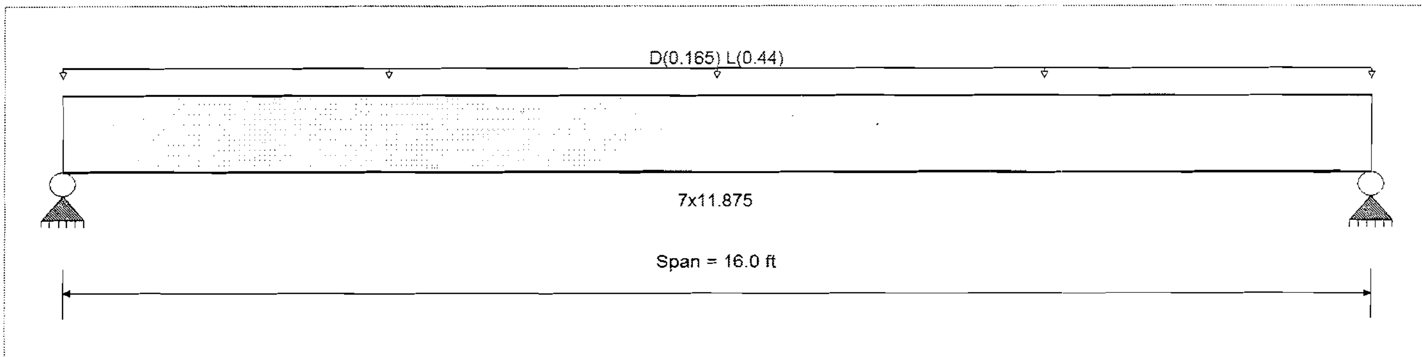
DESCRIPTION: 7. Flush beam over Great Rm

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2900 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - xx	1016.535 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 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 = 0.1650, L = 0.440 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.487: 1	Maximum Shear Stress Ratio	=	0.266 : 1
Section used for this span		**7x11.875**	Section used for this span		**7x11.875**
fb: Actual	=	1,412.12 psi	fv: Actual	=	77.14 psi
Fb: Allowable	=	2,900.00 psi	Fv: Allowable	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.334 in	Ratio =	574 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.459 in	Ratio =	418 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.148	0.081	0.90	1.000	1.00	1.00	1.00	1.00	1.00	5.28	385.12	2610.00	1.17	21.04	261.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.487	0.266	1.00	1.000	1.00	1.00	1.00	1.00	1.00	19.36	1,412.12	2900.00	4.27	77.14	290.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.115	0.063	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.28	385.12	3335.00	1.17	21.04	333.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.319	0.174	1.25	1.000	1.00	1.00	1.00	1.00	1.00	15.84	1,155.37	3625.00	3.50	63.11	362.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.346	0.189	1.15	1.000	1.00	1.00	1.00	1.00	1.00	15.84	1,155.37	3335.00	3.50	63.11	333.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.095	0.052	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.02	439.04	4640.00	1.33	23.98	464.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.258	0.141	1.60	1.000	1.00	1.00	1.00	1.00	1.00	16.39	1,195.81	4640.00	3.62	65.32	464.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING

DESCRIPTION: 7. Flush beam over Great Rm

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.050	0.027	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.17	231.07	4640.00	0.70	12.62	464.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.038	0.021	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.43	177.16	4640.00	0.54	9.68	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4593	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.840	4.840
Overall MINimum	3.520	3.520
D Only	1.320	1.320
+D+L	4.840	4.840
+D+S	1.320	1.320
+D+0.750L	3.960	3.960
+D+0.750L+0.750S	3.960	3.960
+0.60D	0.792	0.792
L Only	3.520	3.520
S Only		

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 8. Dropped beam over Great Rm/Dining Rm

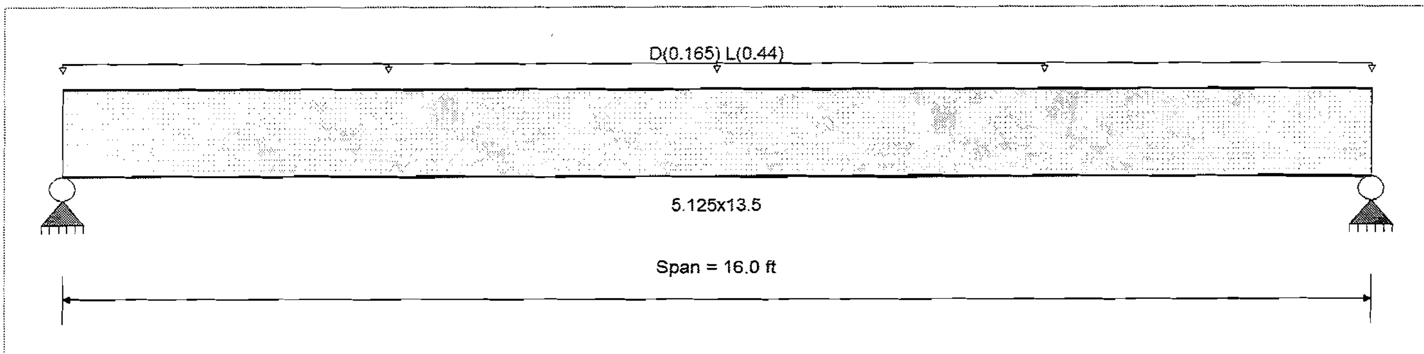
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	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 = 0.1650, L = 0.440 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.622** 1	Maximum Shear Stress Ratio	=	**0.341** : 1
Section used for this span		**5.125x13.5**	Section used for this span		**5.125x13.5**
fb: Actual	=	1,492.37 psi	fv: Actual	=	90.38 psi
Fb: Allowable	=	2,400.00 psi	Fv: Allowable	=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	14.891 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.345 in	Ratio =	556 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.474 in	Ratio =	404 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.188	0.103	0.90	1.000	1.00	1.00	1.00	1.00	1.00	5.28	407.01	2160.00	1.14	24.65	238.50
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.622	0.341	1.00	1.000	1.00	1.00	1.00	1.00	1.00	19.36	1,492.37	2400.00	4.17	90.38	265.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.147	0.081	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.28	407.01	2760.00	1.14	24.65	304.75
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.407	0.223	1.25	1.000	1.00	1.00	1.00	1.00	1.00	15.84	1,221.03	3000.00	3.41	73.95	331.25
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.442	0.243	1.15	1.000	1.00	1.00	1.00	1.00	1.00	15.84	1,221.03	2760.00	3.41	73.95	304.75
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.121	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.02	463.99	3840.00	1.30	28.10	424.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.329	0.181	1.60	1.000	1.00	1.00	1.00	1.00	1.00	16.39	1,263.77	3840.00	3.53	76.54	424.00

Wood Beam Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING

DESCRIPTION: 8. Dropped beam over Great Rm/Dining Rm

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.064	0.035	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.17	244.21	3840.00	0.68	14.79	424.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.049	0.027	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.43	187.22	3840.00	0.52	11.34	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4744	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.840	4.840
Overall MINimum	3.520	3.520
D Only	1.320	1.320
+D+L	4.840	4.840
+D+S	1.320	1.320
+D+0.750L	3.960	3.960
+D+0.750L+0.750S	3.960	3.960
+0.60D	0.792	0.792
L Only	3.520	3.520
S Only		

Wood Beam

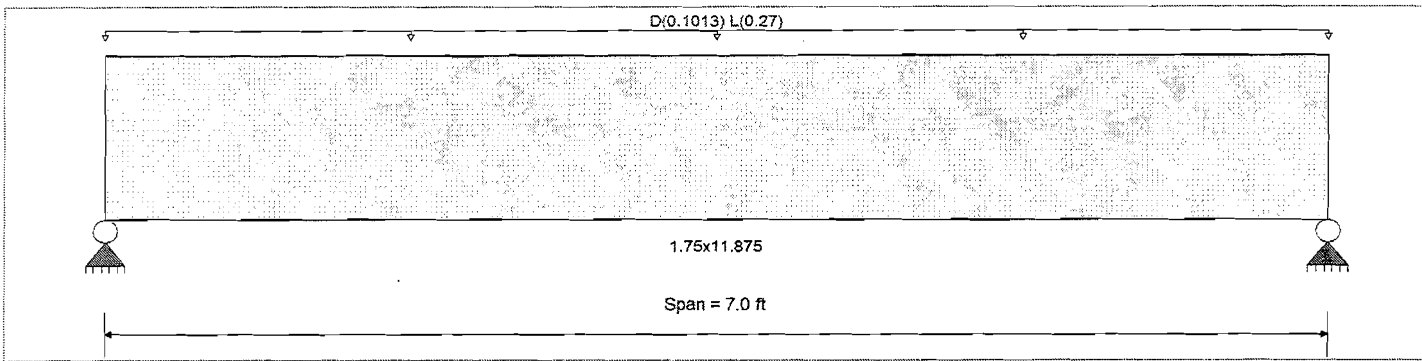
DESCRIPTION: 9. Rim Beam at Stair

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.1013, L = 0.270 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.285 : 1	Maximum Shear Stress Ratio	=	**0.219** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
fb: Actual	=	663.53 psi	fv: Actual	=	67.78 psi
Fb: Allowable	=	2,325.00 psi	Fv: Allowable	=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.039 in	Ratio =	2167 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.053 in	Ratio =	1575 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.087	0.066	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.62	181.03	2092.50	0.26	18.49	279.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.285	0.219	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.27	663.53	2325.00	0.94	67.78	310.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.068	0.052	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.62	181.03	2673.75	0.26	18.49	356.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.187	0.143	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.86	542.90	2906.25	0.77	55.46	387.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.203	0.156	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.86	542.90	2673.75	0.77	55.46	356.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.055	0.043	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.71	206.37	3720.00	0.29	21.08	496.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.151	0.116	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.93	561.91	3720.00	0.80	57.40	496.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 9. Rim Beam at Stair

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.029	0.022	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.37	108.62	3720.00	0.15	11.10	496.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.022	0.017	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.29	83.27	3720.00	0.12	8.51	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0533	3.526		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.300	1.300
Overall MINimum	0.945	0.945
D Only	0.355	0.355
+D+L	1.300	1.300
+D+S	0.355	0.355
+D+0.750L	1.063	1.063
+D+0.750L+0.750S	1.063	1.063
+0.60D	0.213	0.213
L Only	0.945	0.945
S Only		

Project Title:	Chases Corner - Lot 2
Engineer:	Mark Myers, P.E.
Project ID:	
Project Descr:	

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

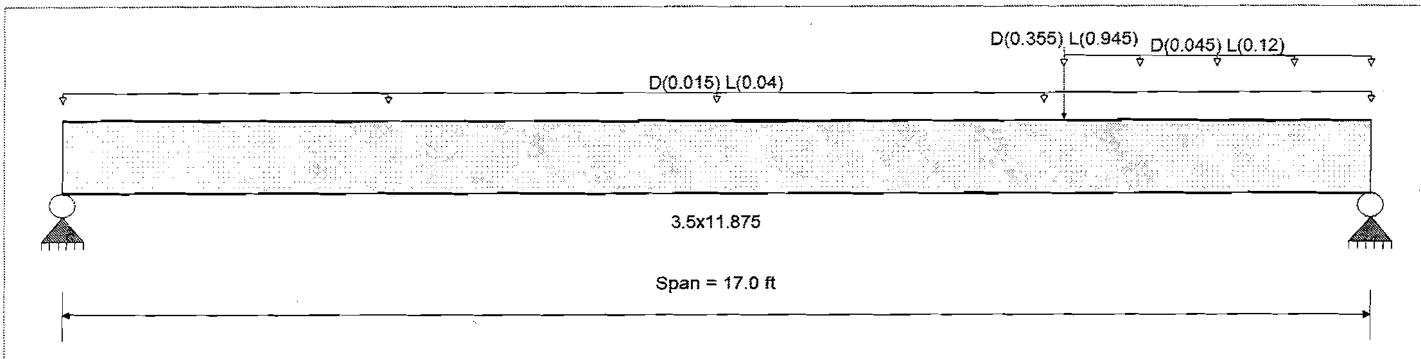
DESCRIPTION: 10. Rim Beam at Stair supporting Bm 9

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.3550, L = 0.9450 k @ 13.0 ft
Uniform Load : D = 0.0450, L = 0.120 k/ft, Extent = 13.0 -->> 17.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.402** 1	Maximum Shear Stress Ratio	=	**0.214** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb: Actual	=	935.30 psi	fv: Actual	=	66.38 psi
Fb: Allowable	=	2,325.00 psi	Fv: Allowable	=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	12.967 ft	Location of maximum on span	=	16.069 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.285 in	Ratio =	714 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.392 in	Ratio =	519 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.122	0.065	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.75	255.28	2092.50	0.50	18.12	279.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.402	0.214	1.00	1.000	1.00	1.00	1.00	1.00	1.00	6.41	935.30	2325.00	1.84	66.38	310.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.095	0.051	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.75	255.28	2673.75	0.50	18.12	356.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.263	0.140	1.25	1.000	1.00	1.00	1.00	1.00	1.00	5.25	765.29	2906.25	1.50	54.31	387.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.286	0.152	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.25	765.29	2673.75	1.50	54.31	356.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.078	0.042	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.99	291.02	3720.00	0.57	20.65	496.00

Wood Beam

DESCRIPTION: 10. Rim Beam at Stair supporting Bm 9

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.213	0.113	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.43	792.10	3720.00	1.56	56.21	496.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.041	0.022	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.05	153.17	3720.00	0.30	10.87	496.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.032	0.017	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.80	117.43	3720.00	0.23	8.33	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.3925	9.245		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.851	2.044
Overall MINimum	0.619	1.486
D Only	0.232	0.558
+D+L	0.851	2.044
+D+S	0.232	0.558
+D+0.750L	0.696	1.672
+D+0.750L+0.750S	0.696	1.672
+0.60D	0.139	0.335
L Only	0.619	1.486
S Only		

Wood Beam

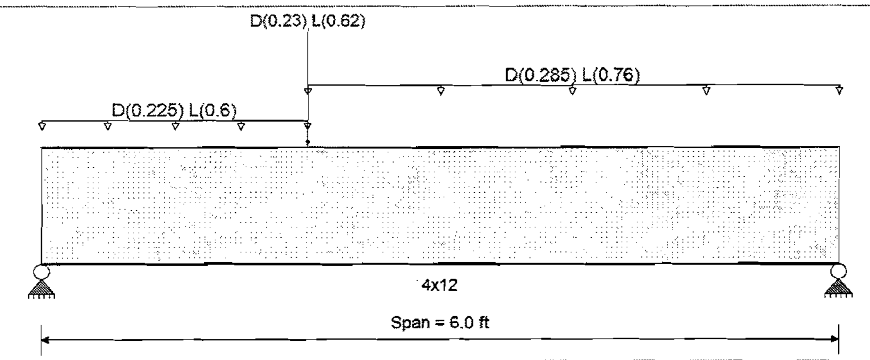
DESCRIPTION: 11. Header at Pantry

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.2250, L = 0.60 k/ft, Extent = 0.0 -->> 2.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.2850, L = 0.760 k/ft, Extent = 2.0 -->> 6.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.230, L = 0.620 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.879**: 1	Maximum Shear Stress Ratio	=	**0.545** : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
fb: Actual	=	870.17 psi	fv: Actual	=	98.14 psi
Fb: Allowable	=	990.00 psi	Fv: Allowable	=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.803 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	0.038 in	Ratio =	1893 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.052 in	Ratio =	1377 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.266	0.165	0.90	1.100	1.00	1.00	1.00	1.00	1.00	1.46	237.00	891.00	0.70	26.72	162.00
+D+L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.879	0.545	1.00	1.100	1.00	1.00	1.00	1.00	1.00	5.35	870.17	990.00	2.58	98.14	180.00
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.208	0.129	1.15	1.100	1.00	1.00	1.00	1.00	1.00	1.46	237.00	1138.50	0.70	26.72	207.00
+D+0.750L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.575	0.357	1.25	1.100	1.00	1.00	1.00	1.00	1.00	4.38	711.88	1237.50	2.11	80.29	225.00
+D+0.750L+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.625	0.388	1.15	1.100	1.00	1.00	1.00	1.00	1.00	4.38	711.88	1138.50	2.11	80.29	207.00
+1.140D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 11. Header at Pantry

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 6.0 ft	1	0.171	0.106	1.60	1.100	1.00	1.00	1.00	1.00	1.00	1.66	270.19	1584.00	0.80	30.46	288.00
+1.105D+0.750L+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.465	0.289	1.60	1.100	1.00	1.00	1.00	1.00	1.00	4.53	736.77	1584.00	2.18	83.09	288.00
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.090	0.056	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.87	142.20	1584.00	0.42	16.03	288.00
+0.460D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.069	0.043	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.67	109.02	1584.00	0.32	12.29	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0523	2.978		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.335	3.345
Overall MINimum	2.427	2.433
D Only	0.908	0.912
+D+L	3.335	3.345
+D+S	0.908	0.912
+D+0.750L	2.728	2.737
+D+0.750L+0.750S	2.728	2.737
+0.60D	0.545	0.547
L Only	2.427	2.433
S Only		

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

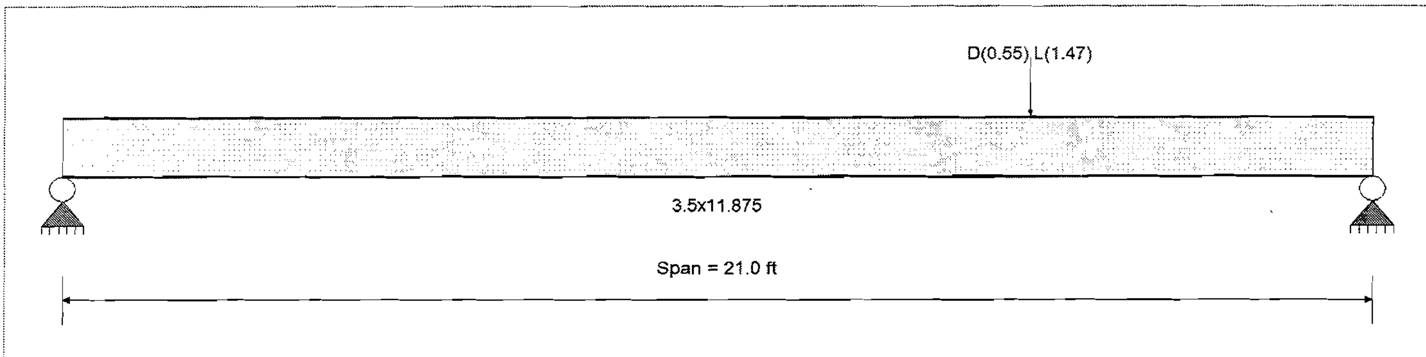
DESCRIPTION: 12. Beam at side by side showers

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Point Load : D = 0.550, L = 1.470 k @ 15.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.514** 1	Maximum Shear Stress Ratio	=	**0.174** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb: Actual	=	1,194.85 psi	fv: Actual	=	53.81 psi
Fb: Allowable	=	2,325.00 psi	Fv: Allowable	=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	15.482 ft	Location of maximum on span	=	15.558 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.472 in	Ratio =	534 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.648 in	Ratio =	388 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.155	0.053	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.23	325.33	2092.50	0.41	14.65	279.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.514	0.174	1.00	1.000	1.00	1.00	1.00	1.00	1.00	8.19	1,194.85	2325.00	1.49	53.81	310.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.122	0.041	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.23	325.33	2673.75	0.41	14.65	356.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.336	0.114	1.25	1.000	1.00	1.00	1.00	1.00	1.00	6.70	977.47	2906.25	1.22	44.02	387.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.366	0.123	1.15	1.000	1.00	1.00	1.00	1.00	1.00	6.70	977.47	2673.75	1.22	44.02	356.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.100	0.034	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.54	370.88	3720.00	0.46	16.70	496.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.272	0.092	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.93	1,011.63	3720.00	1.26	45.56	496.00

Wood Beam

DESCRIPTION: 12. Beam at side by side showers

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.052	0.018	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.34	195.20	3720.00	0.24	8.79	496.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.040	0.014	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.03	149.65	3720.00	0.19	6.74	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.6481	11.726		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.529	1.491
Overall MINimum	0.385	1.085
D Only	0.144	0.406
+D+L	0.529	1.491
+D+S	0.144	0.406
+D+0.750L	0.433	1.220
+D+0.750L+0.750S	0.433	1.220
+0.60D	0.086	0.244
L Only	0.385	1.085
S Only		

Wood Beam

Project File: Chases Lot 2.ec6

DESCRIPTION: 13. Beam over Garage at Grid D

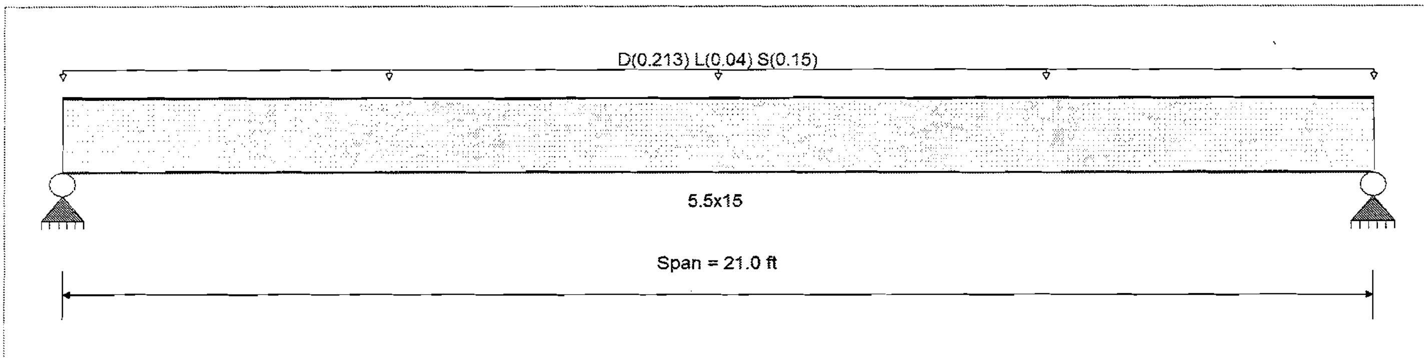
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.2130, L = 0.040, S = 0.150 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.434** 1	Maximum Shear Stress Ratio	=	**0.201** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
fb: Actual	=	1,164.24 psi	fv: Actual	=	61.21 psi
Fb: Allowable	=	2,680.10 psi	Fv: Allowable	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.237 in	Ratio =	1062 >=480	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.574 in	Ratio =	439 >=360	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.326	0.151	0.90	0.971	1.00	1.00	1.00	1.00	1.00	11.74	683.15	2097.47	1.98	35.91	238.50
+D+L					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.348	0.161	1.00	0.971	1.00	1.00	1.00	1.00	1.00	13.95	811.44	2330.52	2.35	42.66	265.00
+D+S					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.434	0.201	1.15	0.971	1.00	1.00	1.00	1.00	1.00	20.01	1,164.24	2680.10	3.37	61.21	304.75
+D+0.750L					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.268	0.124	1.25	0.971	1.00	1.00	1.00	1.00	1.00	13.40	779.37	2913.15	2.25	40.97	331.25
+D+0.750L+0.750S					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.425	0.197	1.15	0.971	1.00	1.00	1.00	1.00	1.00	19.60	1,140.19	2680.10	3.30	59.94	304.75
+1.140D					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.209	0.097	1.60	0.971	1.00	1.00	1.00	1.00	1.00	13.39	778.79	3728.84	2.25	40.94	424.00
+1.105D+0.750L+0.750S					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

DESCRIPTION: 13. Beam over Garage at Grid D

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 21.0 ft	1	0.325	0.150	1.60	0.971	1.00	1.00	1.00	1.00	1.00	20.83	1,211.92	3728.84	3.50	63.71	424.00
+0.60D					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.110	0.051	1.60	0.971	1.00	1.00	1.00	1.00	1.00	7.04	409.89	3728.84	1.19	21.55	424.00
+0.460D					0.971	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 21.0 ft	1	0.084	0.039	1.60	0.971	1.00	1.00	1.00	1.00	1.00	5.40	314.25	3728.84	0.91	16.52	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.5738	10.577		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.812	3.812
Overall MINimum	1.575	1.575
D Only	2.237	2.237
+D+L	2.657	2.657
+D+S	3.812	3.812
+D+0.750L	2.552	2.552
+D+0.750L+0.750S	3.733	3.733
+0.60D	1.342	1.342
L Only	0.420	0.420
S Only	1.575	1.575

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.22.4.26 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

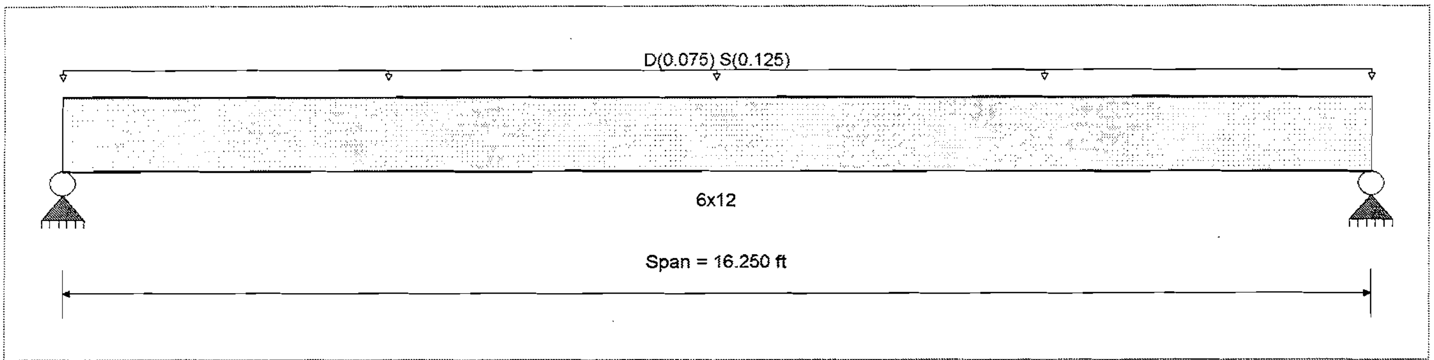
DESCRIPTION: 14. Garage Door Header REV

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0750, S = 0.1250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.649: 1**	Maximum Shear Stress Ratio	=	**0.174 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	653.46 psi	fv: Actual	=	34.04 psi
Fb: Allowable	=	1,006.25 psi	Fv: Allowable	=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.218 in	Ratio =	895 >=480	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.348 in	Ratio =	559 >=360	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.311	0.083	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.48	245.05	787.50	0.54	12.76	153.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.280	0.075	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.48	245.05	875.00	0.54	12.76	170.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.649	0.174	1.15	1.000	1.00	1.00	1.00	1.00	1.00	6.60	653.46	1006.25	1.44	34.04	195.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.224	0.060	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.48	245.05	1093.75	0.54	12.76	212.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.548	0.147	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.57	551.36	1006.25	1.21	28.72	195.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.200	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.82	279.36	1400.00	0.61	14.55	272.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

DESCRIPTION: 14. Garage Door Header REV

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 16.250 ft	1	0.412	0.111	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.83	577.09	1400.00	1.27	30.06	272.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.105	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.49	147.03	1400.00	0.32	7.66	272.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.250 ft	1	0.081	0.022	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.14	112.72	1400.00	0.25	5.87	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.3483	8.184		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.625	1.625
Overall MINimum	1.016	1.016
D Only	0.609	0.609
+D+L	0.609	0.609
+D+S	1.625	1.625
+D+0.750L	0.609	0.609
+D+0.750L+0.750S	1.371	1.371
+0.60D	0.366	0.366
S Only	1.016	1.016

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

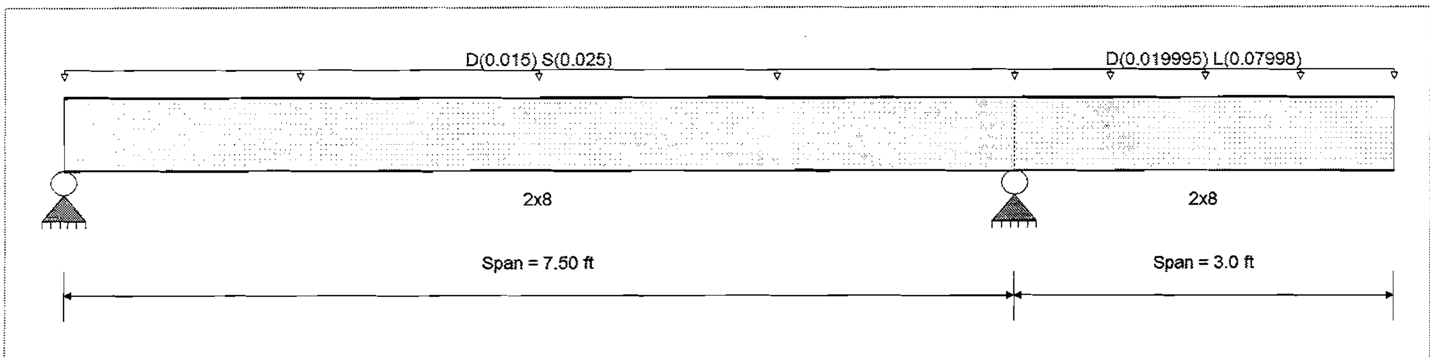
DESCRIPTION: 15. Deck Joists at Upper Cov'd Porch

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.0150, S = 0.0250 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.0150, L = 0.060 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.438 1**	Maximum Shear Stress Ratio	=	**0.275 : 1**
Section used for this span		**2x8**	Section used for this span		**2x8**
fb: Actual	=	410.84 psi	fv: Actual	=	33.05 psi
Fb: Allowable	=	938.40 psi	Fv: Allowable	=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	0.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.103 in	Ratio =	698	>=360	Span: 2 : L Only
Max Upward Transient Deflection	-0.039 in	Ratio =	1858	>=360	Span: 2 : S Only
Max Downward Total Deflection	0.105 in	Ratio =	682	>=240	Span: 2 : +D+L
Max Upward Total Deflection	-0.036 in	Ratio =	1988	>=240	Span: 2 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.097	0.076	0.90	1.200	0.80	1.15	1.00	1.00	1.00	0.09	82.17	844.56	0.06	8.20	108.00
Length = 3.0 ft	2	0.097	0.076	0.90	1.200	0.80	1.15	1.00	1.00	1.00	0.09	82.17	844.56	0.05	8.20	108.00
+D+L					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.438	0.275	1.00	1.200	0.80	1.15	1.00	1.00	1.00	0.45	410.84	938.40	0.24	33.05	120.00
Length = 3.0 ft	2	0.438	0.275	1.00	1.200	0.80	1.15	1.00	1.00	1.00	0.45	410.84	938.40	0.24	33.05	120.00
+D+S					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.201	0.138	1.15	1.200	0.80	1.15	1.00	1.00	1.00	0.24	217.39	1079.16	0.14	19.11	138.00
Length = 3.0 ft	2	0.076	0.138	1.15	1.200	0.80	1.15	1.00	1.00	1.00	0.09	82.17	1079.16	0.05	19.11	138.00
+D+0.750L					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.280	0.176	1.25	1.200	0.80	1.15	1.00	1.00	1.00	0.36	328.67	1173.00	0.19	26.44	150.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 15. Deck Joists at Upper Cov'd Porch

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 3.0 ft	2	0.280	0.176	1.25	1.200	0.80	1.15	1.00	1.00	1.00	0.36	328.67	1173.00	0.19	26.44	150.00
+D+0.750L+0.750S					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.305	0.192	1.15	1.200	0.80	1.15	1.00	1.00	1.00	0.36	328.67	1079.16	0.19	26.44	138.00
Length = 3.0 ft	2	0.305	0.192	1.15	1.200	0.80	1.15	1.00	1.00	1.00	0.36	328.67	1079.16	0.19	26.44	138.00
+1.140D					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.062	0.049	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.10	93.67	1501.44	0.07	9.35	192.00
Length = 3.0 ft	2	0.062	0.049	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.10	93.67	1501.44	0.05	9.35	192.00
+1.105D+0.750L+0.750S					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.225	0.141	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.37	337.30	1501.44	0.20	27.13	192.00
Length = 3.0 ft	2	0.225	0.141	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.37	337.30	1501.44	0.20	27.13	192.00
+0.60D					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.033	0.026	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.05	49.30	1501.44	0.04	4.92	192.00
Length = 3.0 ft	2	0.033	0.026	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.05	49.30	1501.44	0.03	4.92	192.00
+0.460D					1.200	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.50 ft	2	0.025	0.020	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.04	37.80	1501.44	0.03	3.77	192.00
Length = 3.0 ft	2	0.025	0.020	1.60	1.200	0.80	1.15	1.00	1.00	1.00	0.04	37.80	1501.44	0.02	3.77	192.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0395	3.645	L Only	-0.0384	4.358
+D+L	2	0.1055	3.000		0.0000	4.358

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.138	0.416	
Overall MINimum	0.094	0.094	
D Only	0.044	0.128	
+D+L	-0.004	0.416	
+D+S	0.138	0.222	
+D+0.750L	0.008	0.344	
+D+0.750L+0.750S	0.079	0.414	
+0.60D	0.027	0.077	
L Only	-0.048	0.288	
S Only	0.094	0.094	

Wood Beam Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

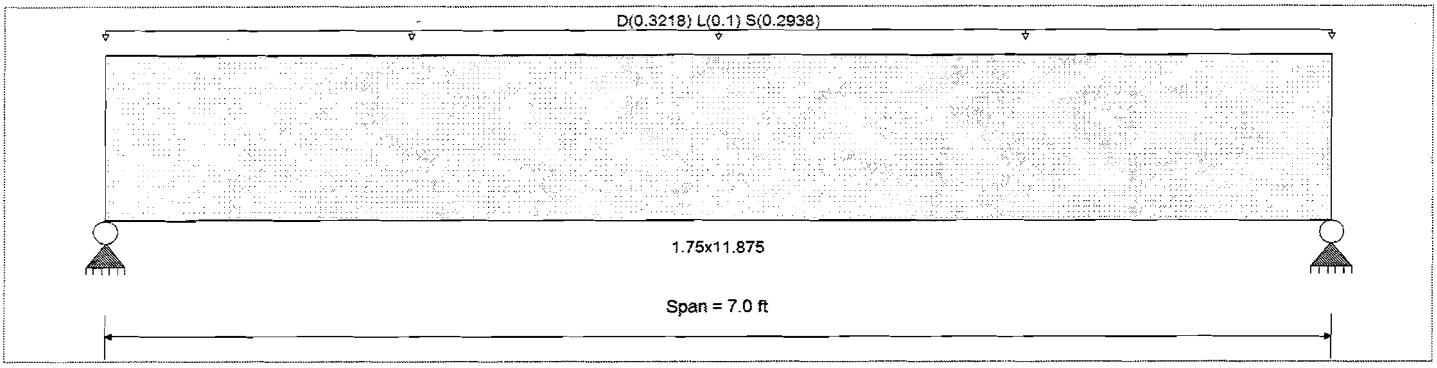
DESCRIPTION: 16. Rim Beam at Grid 4

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.3218, L = 0.10, S = 0.2938 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.412** 1	Maximum Shear Stress Ratio	=	**0.316** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
fb: Actual	=	1,102.87 psi	fv: Actual	=	112.67 psi
Fb: Allowable	=	2,673.75 psi	Fv: Allowable	=	356.50 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.042 in	Ratio =	1991 >=480	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.089 in	Ratio =	948 >=360	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.275	0.211	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.97	575.07	2092.50	0.81	58.75	279.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.324	0.248	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.58	753.77	2325.00	1.07	77.00	310.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.411	0.315	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.77	1,100.10	2673.75	1.56	112.38	356.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.244	0.187	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.43	709.10	2906.25	1.00	72.44	387.50
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.412	0.316	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.78	1,102.87	2673.75	1.56	112.67	356.50
+1.140D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.176	0.135	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.25	655.58	3720.00	0.93	66.97	496.00
+1.105D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.313	0.240	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.99	1,163.25	3720.00	1.65	118.83	496.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 | MYERS ENGINEERING |

DESCRIPTION: 16. Rim Beam at Grid 4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.093	0.071	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.18	345.04	3720.00	0.49	35.25	496.00
+0.460D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.071	0.054	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.91	264.53	3720.00	0.37	27.02	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.0886	3.526		0.0000	0.000

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	2.160	2.160
Overall MINimum	1.028	1.028
D Only	1.126	1.126
+D+L	1.476	1.476
+D+S	2.155	2.155
+D+0.750L	1.389	1.389
+D+0.750L+0.750S	2.160	2.160
+0.60D	0.676	0.676
L Only	0.350	0.350
S Only	1.028	1.028

Wood Beam

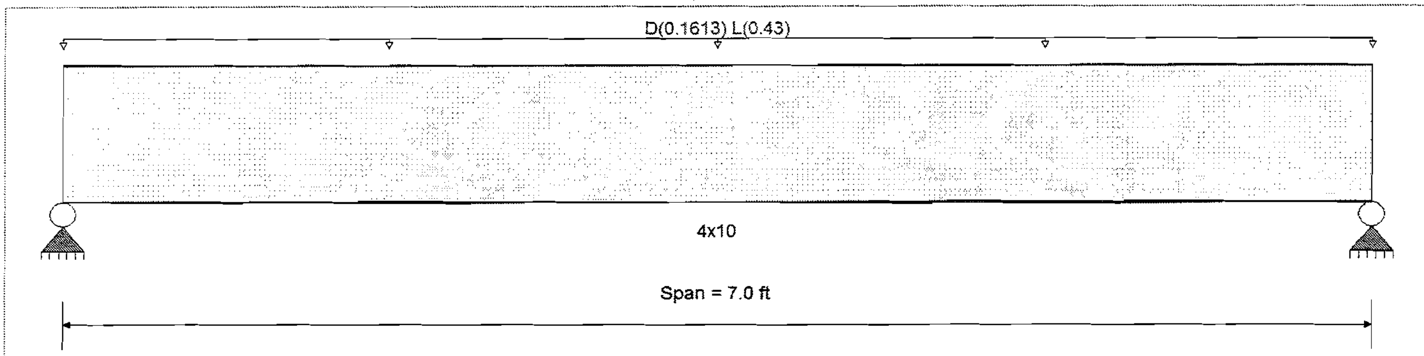
DESCRIPTION: 17. Crawl Beams NOT at brg wall

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.1613, L = 0.430 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.806 1	Maximum Shear Stress Ratio	=	0.416 : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
fb: Actual	=	870.75 psi	fv: Actual	=	74.89 psi
Fb: Allowable	=	1,080.00 psi	Fv: Allowable	=	180.00 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	0.063 in	Ratio =	1327 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.087 in	Ratio =	965 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.244	0.126	0.90	1.200	1.00	1.00	1.00	1.00	1.00	0.99	237.53	972.00	0.44	20.43	162.00
+D+L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.806	0.416	1.00	1.200	1.00	1.00	1.00	1.00	1.00	3.62	870.75	1080.00	1.62	74.89	180.00
+D+S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.191	0.099	1.15	1.200	1.00	1.00	1.00	1.00	1.00	0.99	237.53	1242.00	0.44	20.43	207.00
+D+0.750L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.528	0.272	1.25	1.200	1.00	1.00	1.00	1.00	1.00	2.96	712.45	1350.00	1.32	61.27	225.00
+D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.574	0.296	1.15	1.200	1.00	1.00	1.00	1.00	1.00	2.96	712.45	1242.00	1.32	61.27	207.00
+1.140D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.157	0.081	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.13	270.79	1728.00	0.50	23.29	288.00
+1.105D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.427	0.220	1.60	1.200	1.00	1.00	1.00	1.00	1.00	3.07	737.39	1728.00	1.37	63.42	288.00

Wood Beam

Project File: Chases Lot 2.ec6

LIC# : KW-06015659, Build:20.21.10.30 MYERS ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: 17. Crawl Beams NOT at brg wall

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.082	0.043	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.59	142.52	1728.00	0.26	12.26	288.00
+0.460D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.063	0.033	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.45	109.26	1728.00	0.20	9.40	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0870	3.526		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.070	2.070
Overall MINimum	1.505	1.505
D Only	0.565	0.565
+D+L	2.070	2.070
+D+S	0.565	0.565
+D+0.750L	1.693	1.693
+D+0.750L+0.750S	1.693	1.693
+0.60D	0.339	0.339
L Only	1.505	1.505
S Only		

Wood Beam

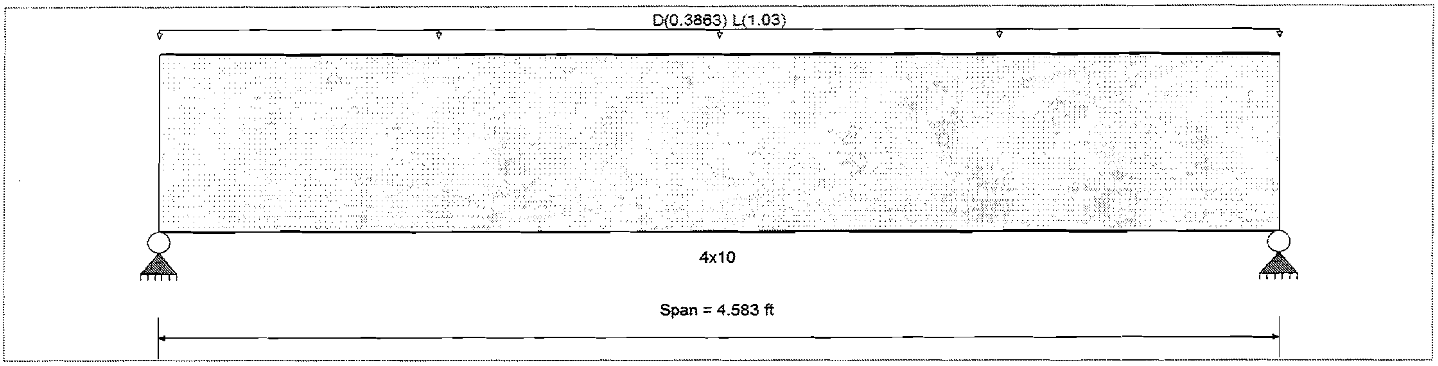
DESCRIPTION: 18. Crawl Beams at brg wall

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.3863, L = 1.030 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.828**: 1	Maximum Shear Stress Ratio	=	**0.555** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
fb: Actual	=	894.02psi	fv: Actual	=	99.88 psi
Fb: Allowable	=	1,080.00psi	Fv: Allowable	=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.292ft	Location of maximum on span	=	3.814 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.028 in	Ratio =	1975 >=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <480	n/a
Max Downward Total Deflection	0.038 in	Ratio =	1436 >=360	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0 <360	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.251	0.168	0.90	1.200	1.00	1.00	1.00	1.00	1.00	1.01	243.85	972.00	0.59	27.24	162.00
+D+L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.828	0.555	1.00	1.200	1.00	1.00	1.00	1.00	1.00	3.72	894.02	1080.00	2.16	99.88	180.00
+D+S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.196	0.132	1.15	1.200	1.00	1.00	1.00	1.00	1.00	1.01	243.85	1242.00	0.59	27.24	207.00
+D+0.750L					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.542	0.363	1.25	1.200	1.00	1.00	1.00	1.00	1.00	3.04	731.47	1350.00	1.76	81.72	225.00
+D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.589	0.395	1.15	1.200	1.00	1.00	1.00	1.00	1.00	3.04	731.47	1242.00	1.76	81.72	207.00
+1.140D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.161	0.108	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.16	277.98	1728.00	0.67	31.06	288.00
+1.105D+0.750L+0.750S					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.438	0.294	1.60	1.200	1.00	1.00	1.00	1.00	1.00	3.15	757.08	1728.00	1.83	84.58	288.00

Wood Beam

DESCRIPTION: 18. Crawl Beams at brg wall

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.085	0.057	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.61	146.31	1728.00	0.35	16.35	288.00
+0.460D					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.583 ft	1	0.065	0.044	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.47	112.17	1728.00	0.27	12.53	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0383	2.308		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.245	3.245
Overall MINimum	2.360	2.360
D Only	0.885	0.885
+D+L	3.245	3.245
+D+S	0.885	0.885
+D+0.750L	2.655	2.655
+D+0.750L+0.750S	2.655	2.655
+0.60D	0.531	0.531
L Only	2.360	2.360
S Only		

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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)

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

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

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

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)