MYERS ENGINEERING

Structural Calculations

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Date: 2023.05.02
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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: Chase's Corner – Lot 1
8908 Southeast 37th Street
Mercer Island, WA

May 2, 2023

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)

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

PROJECT : **Chase's Corner - Lot 1**

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.

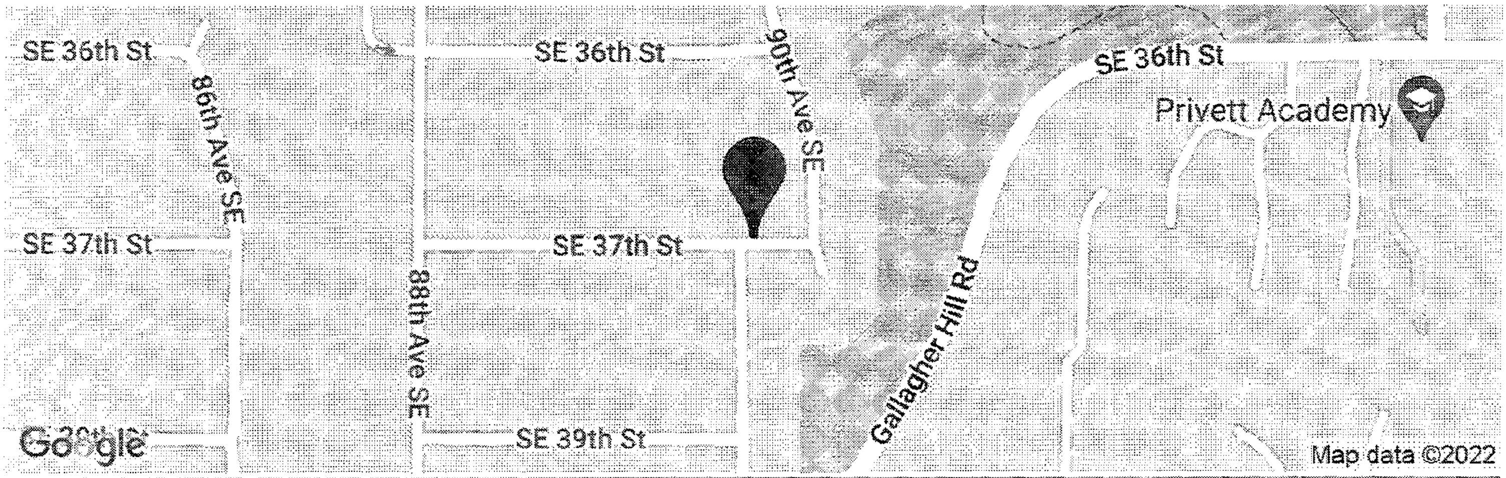
ENGINEERED I-JOISTS

-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

Latitude, Longitude: 47.577, -122.219



Date	4/6/2022, 3:32:40 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}	1.685	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	3.537	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	1.421	Factored deterministic acceleration value. (1.0 second)
PGAd	1.209	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: $S_a := \dfrac{1}{\cos\left(\operatorname{atan}\left(\frac{8}{12}\right)\right)} = 1.2$

Plan Area for Each Level:

$A_1 := 1893\text{ft}^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1689\text{ft}^2$ (Upper Floor) $\quad A_{2b} := 393\text{ft}^2 \cdot S_a$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(42\text{ft}) + 2(58\text{ft})$ (Upper Floor) $\quad P_2 := 2(42\text{ft}) + 2(58\text{ft})$ (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:

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

$$W := w_1 + w_2 = 107391.45 \text{ lb}$$

Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $h_n := 24$ (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) $T_L := 6$ (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: $V_E := C_s \cdot W = 18548.43\,\text{lb}$

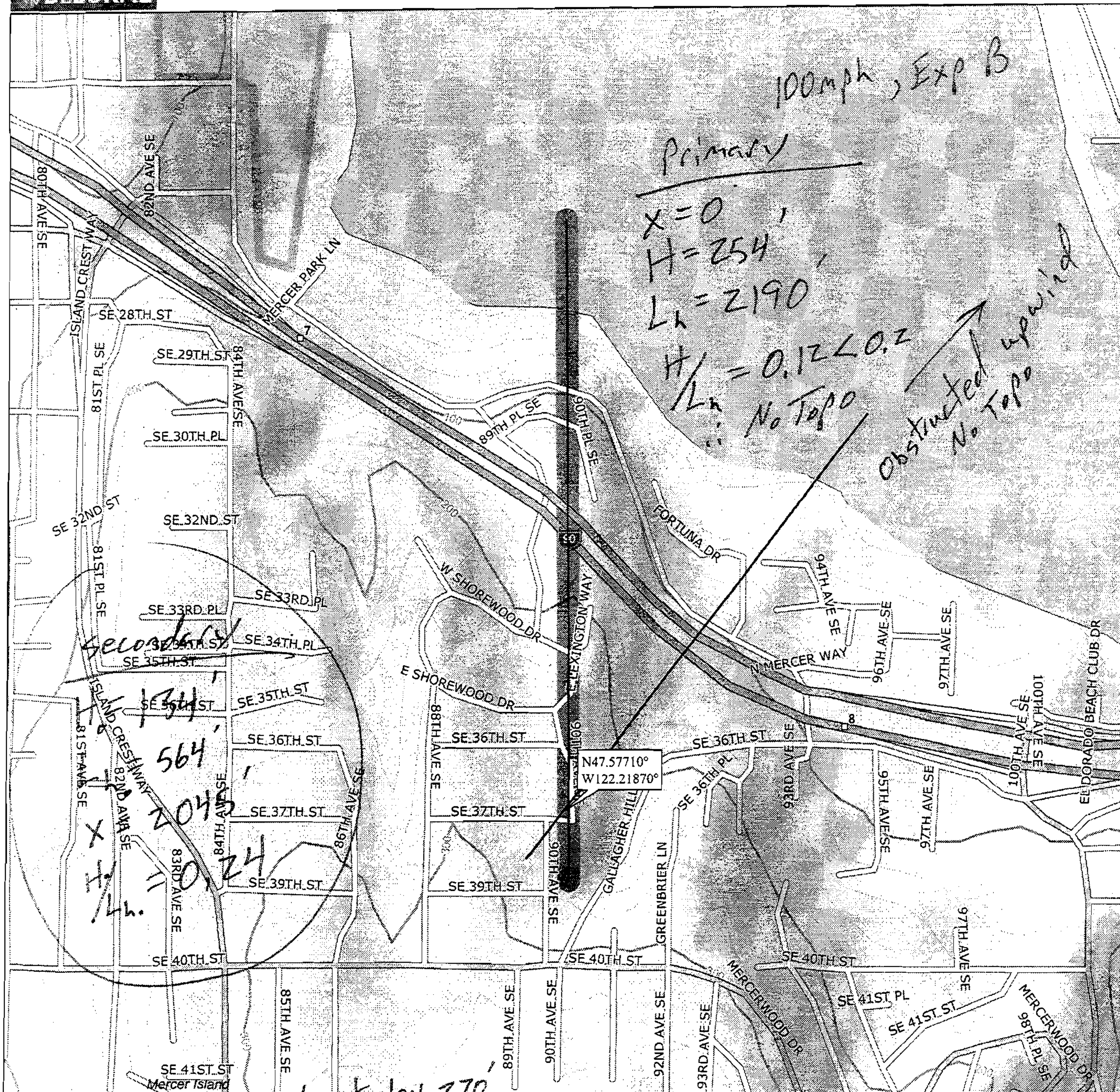
Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

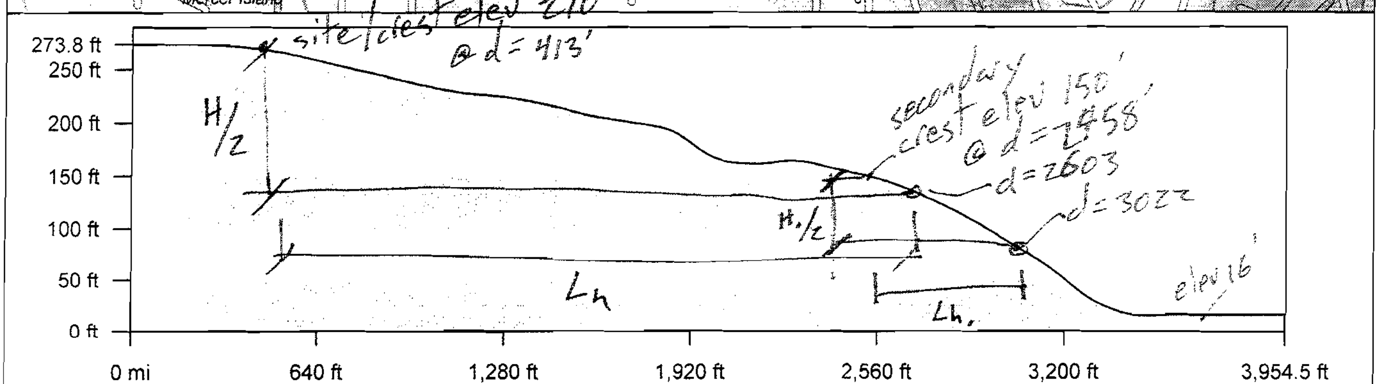
for structures having a period of 0.5 sec or less: $k := 1$

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

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.57$$ $F_1 := C_{v1} \cdot V_E = 10606.33\,\text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.43$$ $F_2 := C_{v2} \cdot V_E = 7942.1\,\text{lb}$ 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

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.

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

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

$G := 0.85$ 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)

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

External Pressure Coefficients w/ Roof Pitch = 3.75/12 (17 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 58ft$ $B_{fb} := 42ft$ $\frac{L_{fb}}{B_{fb}} = 1.38$ $\frac{h}{L_{fb}} = 0.41$

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

$C_{pf2} := 0.08$ Windward Roof

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

$C_{pf4} := -.42$ Leeward Wall

Side to Side:

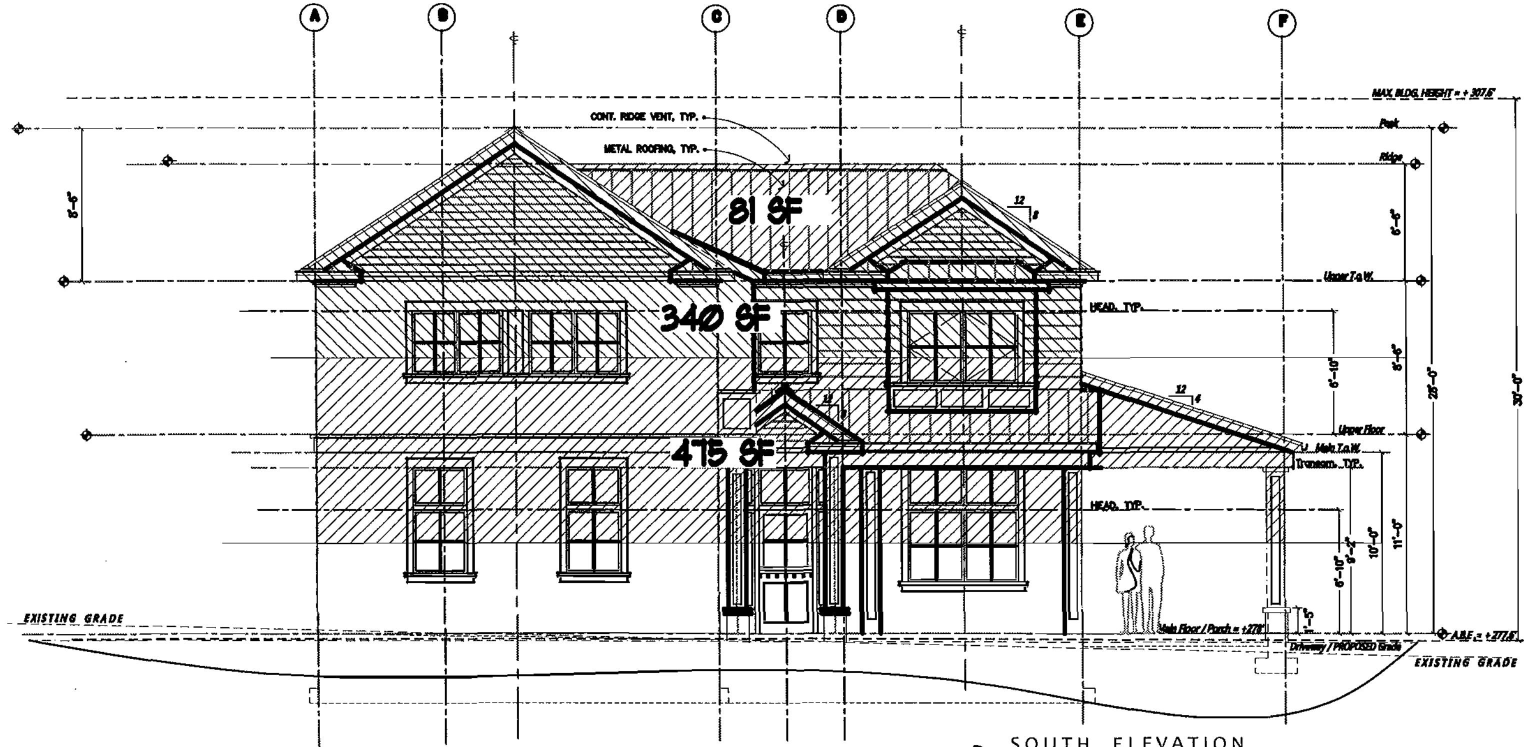
$L_{ss} := 42ft$ $B_{ss} := 58ft$ $\frac{L_{ss}}{B_{ss}} = 0.72$ $\frac{h}{L_{ss}} = 0.57$

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

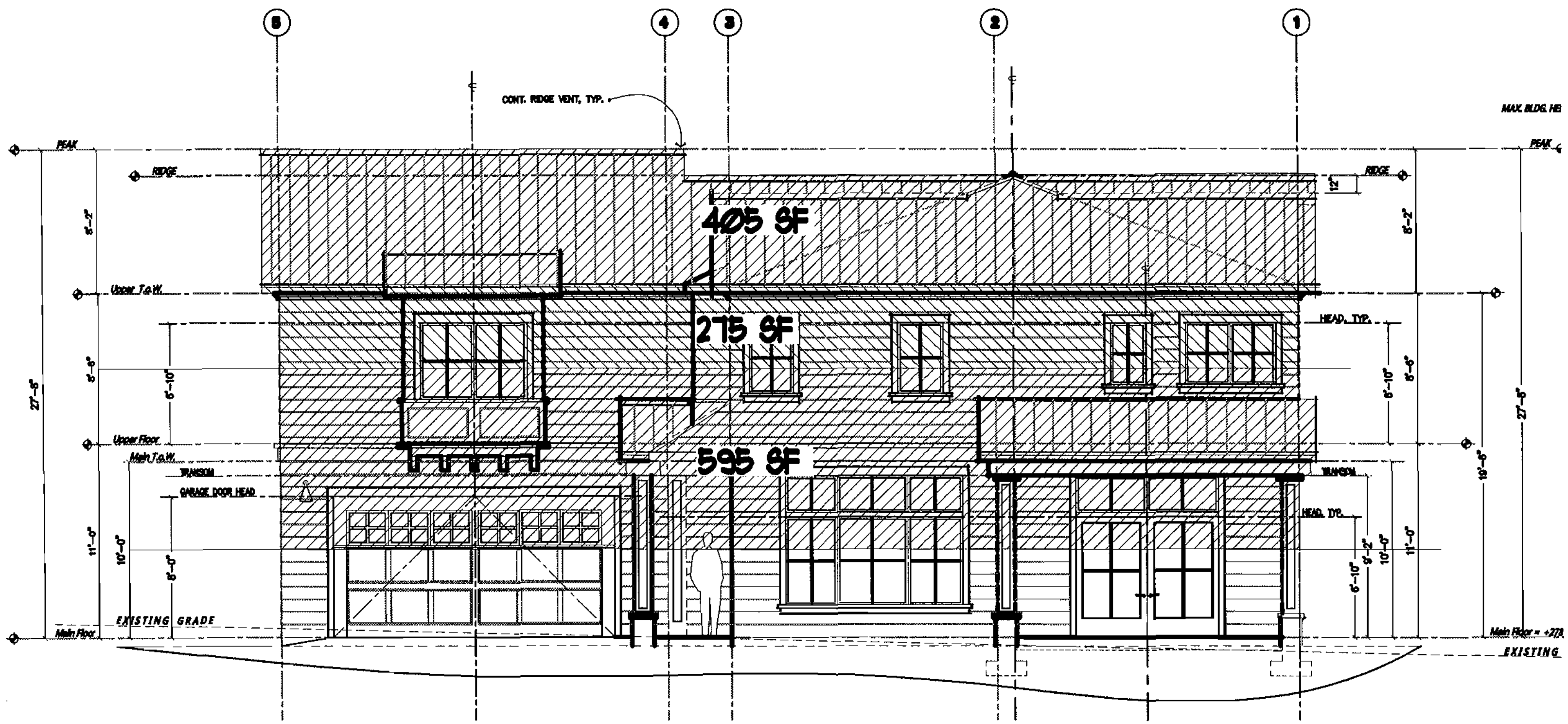
$C_{ps2} := 0.29$ Windward Roof

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

$C_{ps4} := -0.5$ Leeward Wall



2 SOUTH ELEVATION
SCALE: 1/4" = 1'-0"



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

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

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 $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 1\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -7.51\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -5.26\,ft^{-2} \cdot lb$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 3.63\,ft^{-2} \cdot lb$

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 $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -6.26\,ft^{-2} \cdot lb$

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:

$p_{wr1} - p_{lr1} = 8.52\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw1} = 14.63\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw1} = 14.02\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 11.15\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw2} = 15.64\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw2} = 15.02\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$V_{1W} := (p_{wr1} - p_{lr1}) 81 ft^2 + (16 psf) \cdot 340 \cdot ft^2 = 6129.87\,lb$

Wind Pressure at Main Floor (Front to Back):

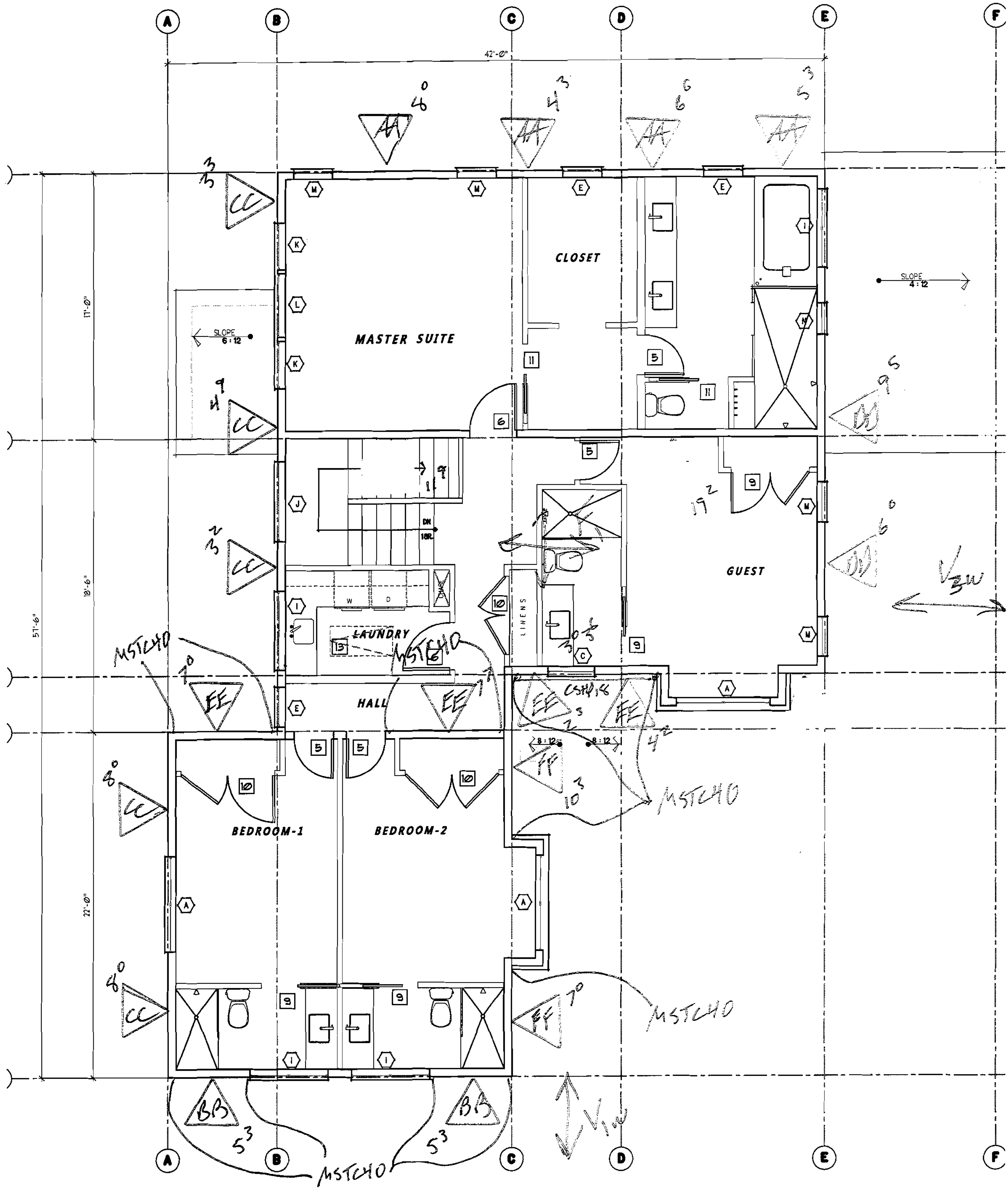
$V_{2W} := (p_{wr1} - p_{lr1}) 0 ft^2 + (16 psf) \cdot 475 \cdot ft^2 = 7600\,lb$

Wind Pressure at Upper Roof (Side to Side):

$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 405 ft^2 + (16 psf) \cdot 275 ft^2 = 8914.58\,lb$

Wind Pressure at Main Floor (Side to Side):

$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0 ft^2 + (16 psf) \cdot 595 ft^2 = 9520\,lb$



NORTH

UPPER FLOOR PLAN

UPPER FLOOR AREA: 1,656.5 S.F.

SCALE: 1/4" = 1'-0"

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WALL AA:

Story Shear due to Wind: $V_{3W} = 8914.58\,\text{lb}$ Story Shear due to Seismic: $F_1 = 10606.33\,\text{lb}$

Bldg Width in direction of Load: $L_t := 57.5 \cdot \text{ft}$ Distance between shear walls: $L_1 := 35.5 \cdot \text{ft}$

Shear Wall Length: $Laa := (8 + 4.25 + 6.5 + 5.25)\text{ft} = 24\,\text{ft}$

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

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

$vaa = 68.8\,\text{ft}^{-1} \cdot \text{lb}$ $\dfrac{vaa}{C_o} = 68.8\,\text{ft}^{-1} \cdot \text{lb}$ $E_{aa} = 95.5\,\text{ft}^{-1} \cdot \text{lb}$ $\dfrac{E_{aa}}{C_o} = 95.5\,\text{ft}^{-1} \cdot \text{lb}$

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

Dead Load Resisting Overturning: $L_{aa} := 4.25 \cdot \text{ft}$ Plate Height: $Pt := 8.5 \cdot \text{ft}$

$W_{aa} := (15 \cdot \text{psf}) \cdot 2 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 0\text{ft}$ $DLRaa := \dfrac{W_{aa} \cdot L_{aa}}{2}$ $DLRaa = 244.37\,\text{lb}$

Chord Force:

$CFaa_w := \dfrac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_w = 584.78\,\text{lb}$ $CFaa_s := \dfrac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_s = 811.71\,\text{lb}$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 438.15\,\text{lb}$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 703.5\,\text{lb}$

No Holdowns Required

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

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

$B_p := \dfrac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 2.37\,\text{ft}$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 1.71\,\text{ft}$

16d @ 16" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vaa} = 20\,\text{ft}$ $\dfrac{(Z_B \cdot C_o)}{E_{aa}} = 14.41\,\text{ft}$

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

WALL BB:

Story Shear due to Wind: $V_{3W} = 8914.58\,lb$ Story Shear due to Seismic: $F_1 = 10606.33\,lb$

Bldg Width in direction of Load: $L_t := 57.5 \cdot ft$ Distance between shear walls: $L_1 := 22 \cdot ft$

Shear Wall Length: $Lbb := (2 \cdot 5.25)ft = 10.5\,ft$

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

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

$vbb = 97.45\,ft^{-1} \cdot lb$ $\frac{vbb}{C_o} = 97.45\,ft^{-1} \cdot lb$ $E_{bb} = 135.27\,ft^{-1} \cdot lb$ $\frac{E_{bb}}{C_o} = 135.27\,ft^{-1} \cdot lb$

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

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

$W_{bb} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $DLRbb = 301.88\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_w = 828.34\,lb$ $CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_s = 1149.79\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 647.21\,lb$ $HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS}) \cdot DLRbb = 1016.11\,lb$

Simpson MSTC40

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 1.67\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 1.21\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vbb} = 14.12\,ft$ $\frac{(Z_B \cdot C_o)}{E_{bb}} = 10.17\,ft$

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

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WALL CC:

Story Shear due to Wind: $V_{1W} = 6129.87\,lb$ Story Shear due to Seismic: $F_1 = 10606.33\,lb$

Bldg Width in direction of Load: $L_t := 42 \cdot ft$ Distance between shear walls: $L_1 := 22 \cdot ft$

Shear Wall Length: $$Lcc := \left[2 \cdot 8 + 3.17\left(\frac{6.33}{8.5}\right) + 4.75 + 3.25\left(\frac{6.5}{8.5}\right)\right] ft = 25.6\,ft$$

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

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

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

$vcc = 37.63\,ft^{-1} \cdot lb$ $\frac{vcc}{C_o} = 37.63\,ft^{-1} \cdot lb$ $E_{cc} = 75.97\,ft^{-1} \cdot lb$ $\frac{E_{cc}}{C_o} = 75.97\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 3.17 \cdot ft$ Plate Height: $Pt := 8.5 \cdot ft$

$W_{cc} := (15 \cdot psf) \cdot 8 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRcc := \frac{W_{cc} \cdot L_{cc}}{2}$ $DLRcc = 324.92\,lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_w = 319.88\,lb$ $CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $CFcc_s = 645.73\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 124.93\,lb$ $HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 501.85\,lb$

No Holdown Required

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 4.34\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 2.15\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vcc} = 36.56\,ft$ $\frac{(Z_B \cdot C_o)}{E_{cc}} = 18.11\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 6129.87\,lb$ Story Shear due to Seismic: $F_1 = 10606.33\,lb$

Bldg Width in direction of Load: $L_t := 42\cdot ft$ Distance between shear walls: $L_1 := 20\cdot ft$

Shear Wall Length: $Ldd := (6 + 9.42)ft = 15.42\,ft$

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

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

$vdd = 56.79\,ft^{-1}\cdot lb$ $\frac{vdd}{C_o} = 56.79\,ft^{-1}\cdot lb$ $E_{dd} = 114.64\,ft^{-1}\cdot lb$ $\frac{E_{dd}}{C_o} = 114.64\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 6\cdot ft$ Plate Height: $Pt := 8.5\cdot ft$

$W_{dd} := (15\cdot psf)\cdot 7\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRdd := \frac{W_{dd}\cdot L_{dd}}{2}$ $DLRdd = 570\,lb$

Chord Force:

$CFdd_w := \frac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_w = 482.71\,lb$ $CFdd_s := \frac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_s = 974.43\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 140.71\,lb$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 722.01\,lb$

No Holdown Required

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

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

$B_P := \frac{(C_D\cdot Z_N\cdot C_o)}{vdd} = 2.87\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{dd}} = 1.42\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vdd} = 24.23\,ft$ $\frac{(Z_B\cdot C_o)}{E_{dd}} = 12\,ft$

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

WALL EE:

Story Shear due to Wind: $V_{3W} = 8914.58\,\text{lb}$ Story Shear due to Seismic: $F_1 = 10606.33\,\text{lb}$

Bldg Width in direction of Load: $L_t := 57.5 \cdot \text{ft}$ Distance between shear walls: $L_1 := 22 \cdot \text{ft}$ $L_2 := 35.5\text{ft}$

Shear Wall Length:

$$Lee := (7 + 7.583 + 2.25 + 4.17)\text{ft} = 21\,\text{ft}$$

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

Wind Force: $vee := \dfrac{\frac{0.6V_{3W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lee}$ Seismic Force: $\rho := 1.0$ $E_{ee} := \dfrac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lee}$

$vee = 127.33\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{vee}{C_o} = 127.33\,\text{ft}^{-1} \cdot \text{lb}$ $E_{ee} = 176.75\,\text{ft}^{-1} \cdot \text{lb}$ $\frac{E_{ee}}{C_o} = 176.75\,\text{ft}^{-1} \cdot \text{lb}$

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

Dead Load Resisting Overturning: $L_{ee} := 7 \cdot \text{ft}$ Plate Height: $Pt := 8.5 \cdot \text{ft}$

$W_{ee} := (15 \cdot \text{psf}) \cdot 2 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 0\text{ft}$ $DLRee := \dfrac{W_{ee} \cdot L_{ee}}{2}$ $DLRee = 402.5\,\text{lb}$

Chord Force:

$CFee_w := \dfrac{vee \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}}$ $CFee_w = 1082.33\,\text{lb}$ $CFee_s := \dfrac{E_{ee} \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}}$ $CFee_s = 1502.35\,\text{lb}$

Holdown Force:

$HDFee_w := CFee_w - 0.6 \cdot DLRee = 840.83\,\text{lb}$ $HDFee_s := CFee_s - (0.6 - 0.14S_{DS})DLRee = 1324.11\,\text{lb}$

Simpson MSTC40

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

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

$B_n := \dfrac{(Z_N \cdot C_D \cdot C_o)}{vee} = 1.28\,\text{ft}$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{ee}} = 0.92\,\text{ft}$

16d @ 8" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vee} = 10.81\,\text{ft}$ $\dfrac{(Z_B \cdot C_o)}{E_{ee}} = 7.79\,\text{ft}$

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

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PROJECT : **Chase's Corner - Lot 1**

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WALL FF:

Story Shear due to Wind: $V_{1W} = 6129.87\,lb$

Story Shear due to Seismic: $F_1 = 10606.33\,lb$

Bldg Width in direction of Load: $L_t := 42 \cdot ft$

Distance between shear walls: $L_1 := 22 \cdot ft$ $\quad L_2 := 20ft$

Shear Wall Length:

$$Lff := (10.25 + 7)ft = 17.25\,ft$$

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

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

Wind Force: $vff := \dfrac{\frac{0.6V_{1W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lff}$

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

$vff = 106.61\,ft^{-1} \cdot lb$ $\qquad \dfrac{vff}{C_o} = 106.61\,ft^{-1} \cdot lb$

$E_{ff} = 215.2\,ft^{-1} \cdot lb$ $\qquad \dfrac{E_{ff}}{C_o} = 215.2\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{ff} := 7 \cdot ft$

Plate Height: $Pt := 8.5 \cdot ft$

$W_{ff} := (15 \cdot psf) \cdot 12 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRff := \dfrac{W_{ff} \cdot L_{ff}}{2}$ $\qquad DLRff = 927.5\,lb$

Chord Force:

$CFff_w := \dfrac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $\qquad CFff_w = 906.15\,lb$

$CFff_s := \dfrac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $\qquad CFff_s = 1829.21\,lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6 \cdot DLRff = 349.65\,lb$

$HDFff_s := CFff_s - \left(0.6 - 0.14S_{DS}\right)DLRff = 1418.49\,lb$

Simpson MSTC40

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

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

$B_s := \dfrac{\left(Z_N \cdot C_D \cdot C_o\right)}{vff} = 1.53\,ft$ $\qquad \dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_{ff}} = 0.76\,ft$

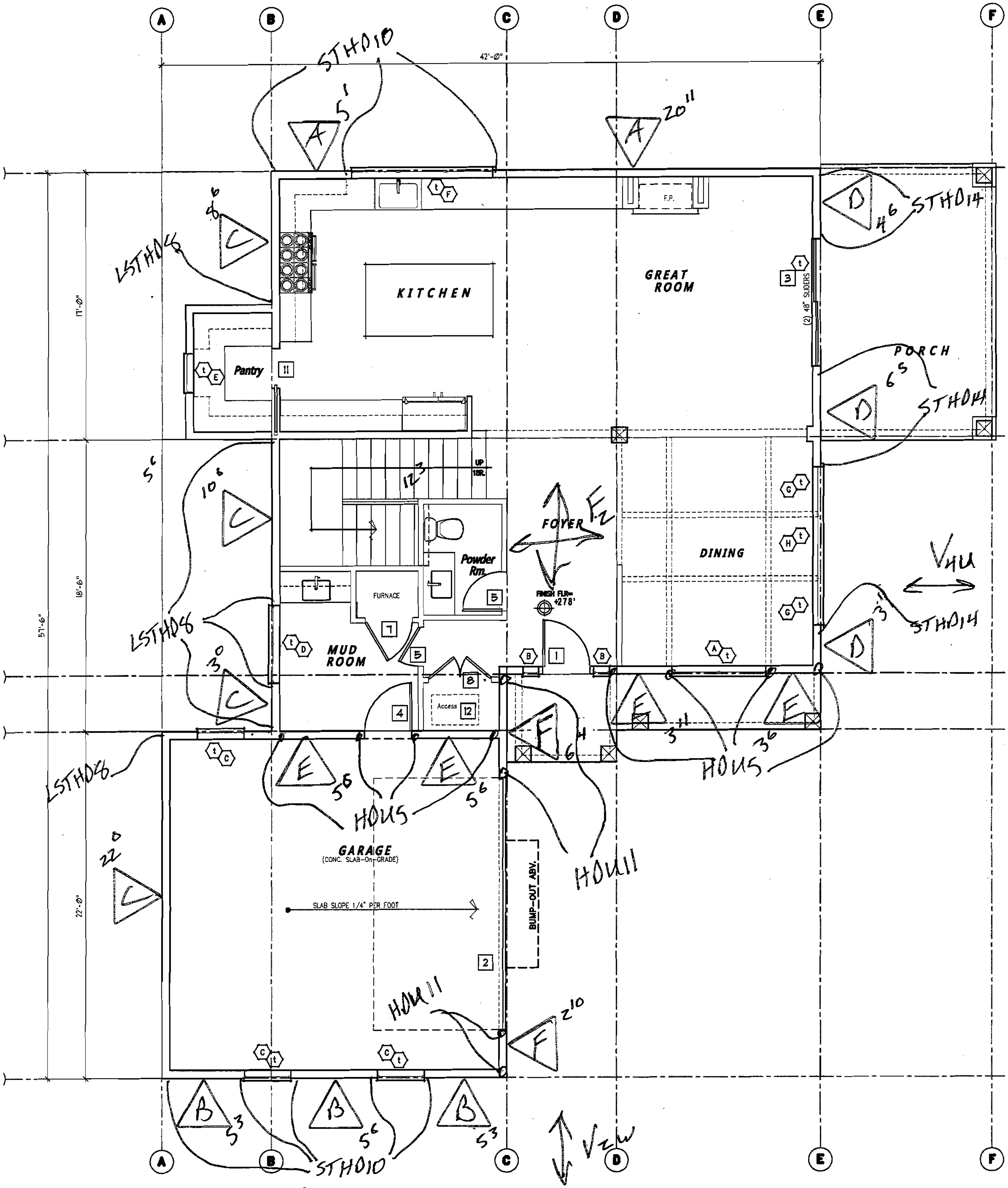
16d @ 8" o.c.

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

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

$As := \dfrac{\left(Z_B \cdot C_o\right)}{vff} = 12.91\,ft$ $\qquad \dfrac{\left(Z_B \cdot C_o\right)}{E_{ff}} = 6.39\,ft$

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



MAIN FLOOR PLAN

MAIN FLOOR AREA: 1,219.5 S.F.

GARAGE AREA: 484 S.F.

SCALE: 1/4" = 1'-0"

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WALL A:

Story Shear due to Wind: $V_{4W} = 9520\,lb$ Story Shear due to Seismic: $F_2 = 7942.1\,lb$

Bldg Width in direction of Load: $L_t := 57.5 \cdot ft$ Distance between shear walls: $L_1 := 35.5 \cdot ft$

Shear Wall Length: $La := (5.083 + 20.917)ft = 26\,ft$

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: $\rho := 1.0$ $E_a := \dfrac{E_{aa} \cdot Laa + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$va = 131.32\,ft^{-1} \cdot lb$ $\frac{va}{C_o} = 131.32\,ft^{-1} \cdot lb$ $E_a = 154.16\,ft^{-1} \cdot lb$ $\frac{E_a}{C_o} = 154.16\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_a := 5.083 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_a := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 8.5ft$ $DLRa := \frac{W_a \cdot L_a}{2}$ $DLRa = 470.18\,lb$

Chord Force:

$CFa_w := \frac{va \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_w = 1313.23\,lb$ $CFa_w + CFaa_w = 1898.01\,lb$

$CFa_s := \frac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $CFa_s = 1541.57\,lb$ $CFa_s + CFaa_s = 2353.28\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 1031.13\,lb$

$HDFa_w + HDFaa_w = 1469.28\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS}) \cdot DLRa = 1333.36\,lb$

$HDFa_s + HDFaa_s = 2036.85\,lb$

Simpson STHD10/RJ

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{va} = 1.24\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 1.06\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{va} = 10.48\,ft$ $\frac{(Z_B \cdot C_o)}{E_a} = 8.93\,ft$

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

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WALL B:

Story Shear due to Wind: $V_{4W} = 9520\,lb$

Story Shear due to Seismic: $F_2 = 7942.1\,lb$

Bldg Width in direction of Load: $L_t := 57.5 \cdot ft$

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

Shear Wall Length: $Lb := (2 \cdot 5.25 + 5.5)ft = 16\,ft$

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

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

$vb = 132.25\,ft^{-1} \cdot lb$ $\quad \frac{vb}{C_o} = 132.25\,ft^{-1} \cdot lb$

$E_b = 155.24\,ft^{-1} \cdot lb$ $\quad \frac{E_b}{C_o} = 155.24\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_b := 5.25 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $\quad DLRb := \frac{W_b \cdot L_b}{2}$ $\quad DLRb = 288.75\,lb$

Chord Force:

$CFb_w := \frac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $\quad CFb_w = 1322.48\,lb$

$CFb_s := \frac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $\quad CFb_s = 1552.42\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1149.23\,lb$

$HDFb_w + HDFbb_w = 1796.44\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14 S_{DS}) \cdot DLRb = 1424.56\,lb$

$HDFb_s + HDFbb_s = 2440.67\,lb$

Simpson STHD10/RJ

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vb} = 1.23\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 1.05\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vb} = 10.4\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_b} = 8.86\,ft$

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

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WALL C:

Story Shear due to Wind: $V_{2W} = 7600\,\text{lb}$ Story Shear due to Seismic: $F_2 = 7942.1\,\text{lb}$

Bldg Width in direction of Load: $L_t := 42\cdot\text{ft}$ Distance between shear walls: $L_1 := 22\cdot\text{ft}$

Shear Wall Length: $Lc := (22 + 10.5 + 8.5)\text{ft} = 41\,\text{ft}$

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

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

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

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

$vc = 52.62\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{vc}{C_o} = 52.62\,\text{ft}^{-1}\cdot\text{lb}$ $E_c = 82.94\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_c}{C_o} = 82.94\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_c := 8.5\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_c := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 1\text{ft}$ $DLRc := \frac{W_c\cdot L_c}{2}$ $DLRc = 467.5\,\text{lb}$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_w = 526.23\,\text{lb}$ $CFc_w + CFcc_w = 846.12\,\text{lb}$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_s = 829.4\,\text{lb}$ $CFc_s + CFcc_s = 1475.13\,\text{lb}$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 245.73\,\text{lb}$
$HDFc_w + HDFcc_w = 370.66\,\text{lb}$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 622.38\,\text{lb}$
$HDFc_s + HDFcc_s = 1124.23\,\text{lb}$

Simpson LSTHD8

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 3.1\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 1.97\,\text{ft}$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vc} = 26.15\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_c} = 16.59\,\text{ft}$

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

WALL D:

Story Shear due to Wind: $V_{2W} = 7600\,\text{lb}$

Story Shear due to Seismic: $F_2 = 7942.1\,\text{lb}$

Bldg Width in direction of Load: $L_t := 42\cdot\text{ft}$

Distance between shear walls: $L_1 := 20\cdot\text{ft}$

Shear Wall Length: $Ld := \left[4.5\left(\frac{9}{10}\right) + 6.42 + 3.083\left(\frac{6.1267}{10}\right)\right]\text{ft} = 12.36\,\text{ft}$

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

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

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

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

$vd = 158.7\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \dfrac{vd}{C_o} = 158.7\,\text{ft}^{-1}\cdot\text{lb}$

$E_d = 250.14\,\text{ft}^{-1}\cdot\text{lb}$ $\quad \dfrac{E_d}{C_o} = 250.14\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_d := 3.083\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

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

$DLRd := \dfrac{W_d\cdot L_d}{2}$ $\quad DLRd = 169.56\,\text{lb}$

Chord Force:

$CFd_w := \dfrac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $\quad CFd_w = 1587.05\,\text{lb}$

$CFd_w + CFdd_w = 2069.76\,\text{lb}$

$CFd_s := \dfrac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $\quad CFd_s = 2501.37\,\text{lb}$

$CFd_s + CFdd_s = 3475.79\,\text{lb}$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 1485.31\,\text{lb}$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 2426.28\,\text{lb}$

$HDFd_w + HDFdd_w = 1626.02\,\text{lb}$

$HDFd_s + HDFdd_s = 3148.29\,\text{lb}$

Simpson STHD14/RJ

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

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

$B_R := \dfrac{(C_D\cdot Z_N\cdot C_o)}{vd} = 1.03\,\text{ft}$ $\quad \dfrac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 0.65\,\text{ft}$

16d @ 8" o.c.

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

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

$As := \dfrac{(Z_B\cdot C_o)}{vd} = 8.67\,\text{ft}$ $\quad \dfrac{(Z_B\cdot C_o)}{E_d} = 5.5\,\text{ft}$

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

WALL E:

Story Shear due to Wind: $V_{4W} = 9520\,lb$ Story Shear due to Seismic: $F_2 = 7942.1\,lb$

Bldg Width in direction of Load: $L_t := 57.5 \cdot ft$ Distance between shear walls: $L_1 := 22 \cdot ft$ $L_2 := 35.5ft$

Shear Wall Length: $Le := \left[2 \cdot 5.5 + 3.917\left(\frac{7.83}{10}\right) + 3.5\left(\frac{7}{10}\right)\right]ft = 16.52\,ft$

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

Wind Force: $ve := \dfrac{vee \cdot Lee + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le}$ Seismic Force: $\rho := 1.0$ $E_e := \dfrac{E_{ee} \cdot Lee + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le}$

$ve = 334.83\,ft^{-1} \cdot lb$ $\frac{ve}{C_o} = 334.83\,ft^{-1} \cdot lb$ $E_e = 393.05\,ft^{-1} \cdot lb$ $\frac{E_e}{C_o} = 393.05\,ft^{-1} \cdot lb$

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

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

$W_e := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 7.5ft$ $DLRe := \frac{W_e \cdot L_e}{2}$ $DLRe = 358.75\,lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_w = 3348.29\,lb$ $CFe_w + CFee_w = 4430.62\,lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_s = 3930.46\,lb$ $CFe_s + CFee_s = 5432.81\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 3133.04\,lb$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS}) \cdot DLRe = 3771.6\,lb$

$HDFe_w + HDFee_w = 3973.87\,lb$ $HDFe_s + HDFee_s = 5095.71\,lb$

Simpson HDU5 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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{ve} = 0.49\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 0.42\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{ve} = 4.11\,ft$ $\frac{(Z_B \cdot C_o)}{E_e} = 3.5\,ft$

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

WALL F:

Story Shear due to Wind: $V_{2W} = 7600\,lb$

Story Shear due to Seismic: $F_2 = 7942.1\,lb$

Bldg Width in direction of Load: $L_t := 42\cdot ft$

Distance between shear walls: $L_1 := 22\cdot ft$ $L_2 := 20ft$

Shear Wall Length: $Lf := (6.33 + 2.875)ft = 9.21\,ft$

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

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

Wind Force: $vf := \dfrac{vff\cdot Lff + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lf}$

Seismic Force: $\rho := 1.0$ $E_f := \dfrac{E_{ff}\cdot Lff + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lf}$

$vf = 447.47\,ft^{-1}\cdot lb$ $\frac{vf}{C_o} = 447.47\,ft^{-1}\cdot lb$

$E_f = 705.26\,ft^{-1}\cdot lb$ $\frac{E_f}{C_o} = 705.26\,ft^{-1}\cdot lb$

P1-2: 7/16" Sheathing w/ 10d nails @ 2" O.C.
Wind Capacity = 1002 plf
Seismic Capacity = 716 plf

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

$W_f := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$

$DLRf := \frac{W_f\cdot L_f}{2}$ $DLRf = 158.13\,lb$

Chord Force:

$CFf_w := \frac{vf\cdot L_f\cdot Pt}{C_o\cdot L_f}$ $CFf_w = 4474.7\,lb$

$CFf_w + CFff_w = 5380.85\,lb$

$CFf_s := \frac{E_f\cdot L_f\cdot Pt}{C_o\cdot L_f}$ $CFf_s = 7052.64\,lb$

$CFf_s + CFff_s = 8881.84\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6\cdot DLRf = 4379.82\,lb$

$HDFf_w + HDFff_w = 4729.48\,lb$

$HDFf_s := CFf_s - (0.6 - 0.14S_{DS})\cdot DLRf = 6982.61\,lb$

$HDFf_s + HDFff_s = 8401.1\,lb$

Simpson HDU11 w/ PAB8 anchor embedded 8" into enlarged footing (24" wide minimum)

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vf} = 0.36\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_f} = 0.23\,ft$

16d @ 3" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vf} = 3.08\,ft$ $\frac{(Z_B\cdot C_o)}{E_f} = 1.95\,ft$

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

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$vaa \cdot \frac{Laa}{35ft} = 47.18\,ft^{-1} \cdot lb$ $\quad E_{aa} \cdot \frac{Laa}{35ft} = 65.48\,ft^{-1} \cdot lb$

Wall Lines DD:

$vdd \cdot \frac{Ldd}{35ft} = 25.02\,ft^{-1} \cdot lb$ $\quad E_{dd} \cdot \frac{Ldd}{35ft} = 50.51\,ft^{-1} \cdot lb$

Wall Lines BB:

$vbb \cdot \frac{Lbb}{22ft} = 46.51\,ft^{-1} \cdot lb$ $\quad E_{bb} \cdot \frac{Lbb}{22ft} = 64.56\,ft^{-1} \cdot lb$

Wall Lines EE:

$vee \cdot \frac{Lee}{35ft} = 76.41\,ft^{-1} \cdot lb$ $\quad E_{ee} \cdot \frac{Lee}{35ft} = 106.06\,ft^{-1} \cdot lb$

Wall Lines CC:

$vcc \cdot \frac{Lcc}{57ft} = 16.9\,ft^{-1} \cdot lb$ $\quad E_{cc} \cdot \frac{Lcc}{57ft} = 34.11\,ft^{-1} \cdot lb$

Wall Lines FF:

$vff \cdot \frac{Lff}{57ft} = 32.26\,ft^{-1} \cdot lb$ $\quad E_{ff} \cdot \frac{Lff}{57ft} = 65.13\,ft^{-1} \cdot lb$

Wall Lines A:

$\frac{va \cdot La - vaa \cdot Laa}{35ft} = 50.38\,ft^{-1} \cdot lb$ $\quad \frac{E_a \cdot La - E_{aa} \cdot Laa}{35ft} = 49.03\,ft^{-1} \cdot lb$ $\quad \frac{va \cdot La}{35ft} = 97.55\,ft^{-1} \cdot lb$ $\quad \frac{E_a \cdot La}{35ft} = 114.52\,ft^{-1} \cdot lb$

Wall Lines B:

$\frac{vb \cdot Lb - vbb \cdot Lbb}{22ft} = 49.67\,ft^{-1} \cdot lb$ $\quad \frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{22ft} = 48.34\,ft^{-1} \cdot lb$ $\quad \frac{vb \cdot Lb}{22ft} = 96.18\,ft^{-1} \cdot lb$ $\quad \frac{E_b \cdot Lb}{22ft} = 112.9\,ft^{-1} \cdot lb$

Wall Lines C:

$\frac{vc \cdot Lc - vcc \cdot Lcc}{49ft} = 24.37\,ft^{-1} \cdot lb$ $\quad \frac{E_c \cdot Lc - E_{cc} \cdot Lcc}{49ft} = 29.72\,ft^{-1} \cdot lb$ $\quad \frac{vc \cdot Lc}{49ft} = 44.03\,ft^{-1} \cdot lb$ $\quad \frac{E_c \cdot Lc}{49ft} = 69.4\,ft^{-1} \cdot lb$

Wall Lines D:

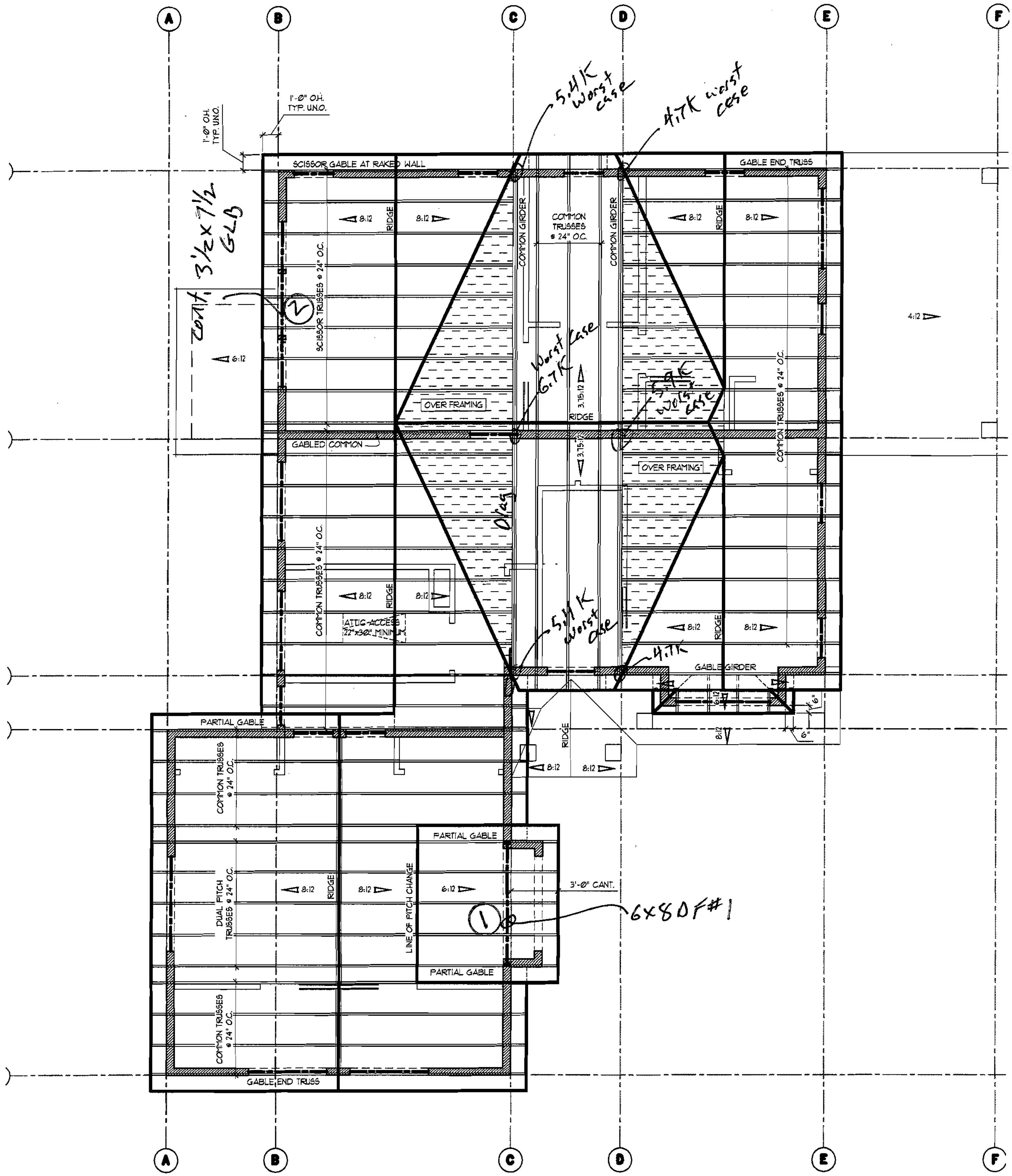
$\frac{vd \cdot Ld - vdd \cdot Ldd}{35ft} = 31.02\,ft^{-1} \cdot lb$ $\quad \frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{35ft} = 37.82\,ft^{-1} \cdot lb$ $\quad \frac{vd \cdot Ld}{35ft} = 56.04\,ft^{-1} \cdot lb$ $\quad \frac{E_d \cdot Ld}{35ft} = 88.33\,ft^{-1} \cdot lb$

Wall Line E:

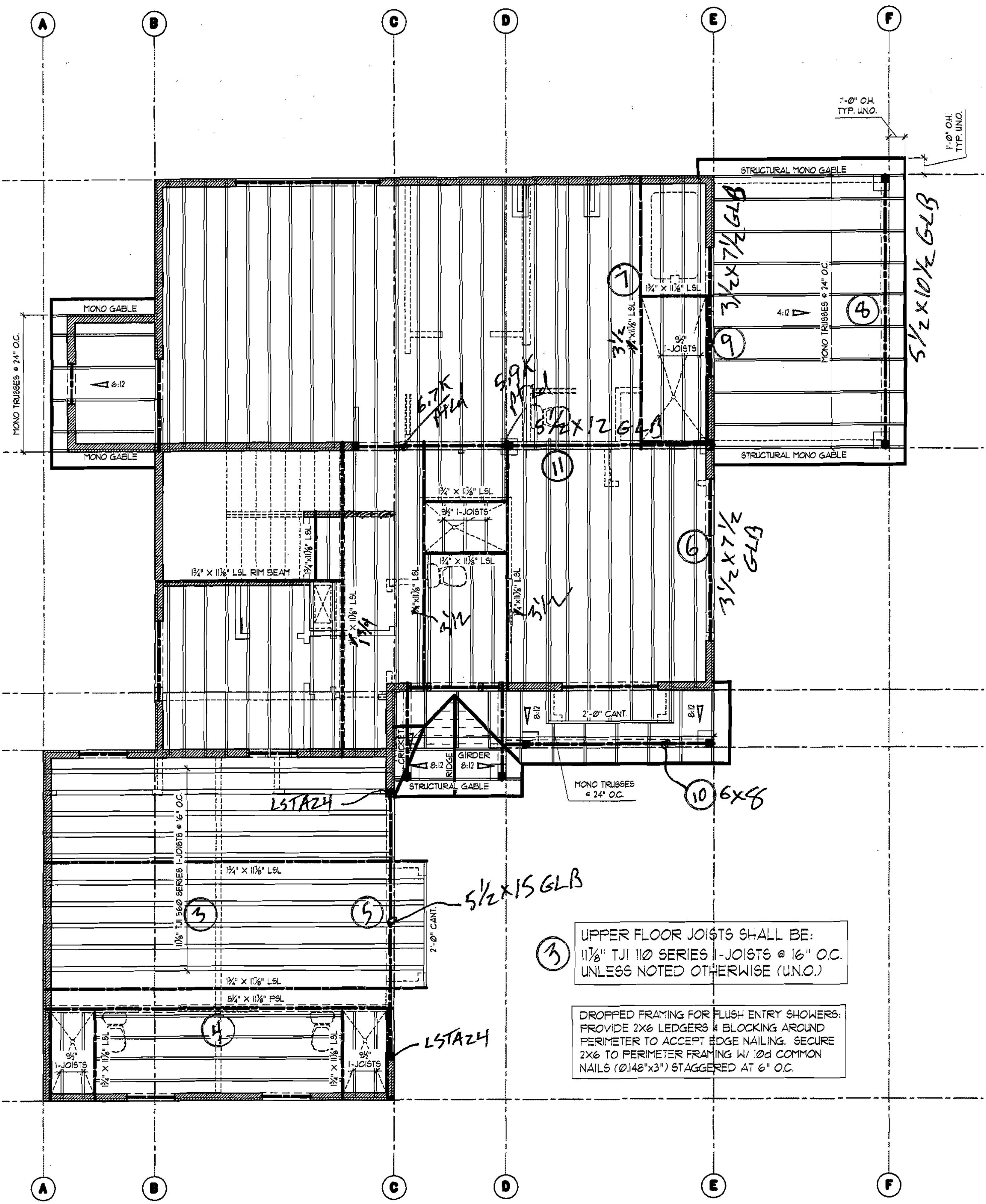
$\frac{ve \cdot Le - vee \cdot Lee}{42ft} = 68\,ft^{-1} \cdot lb$ $\quad \frac{E_e \cdot Le - E_{ee} \cdot Lee}{42ft} = 66.18\,ft^{-1} \cdot lb$ $\quad \frac{ve \cdot Le}{42ft} = 131.68\,ft^{-1} \cdot lb$ $\quad \frac{E_e \cdot Le}{42ft} = 154.57\,ft^{-1} \cdot lb$

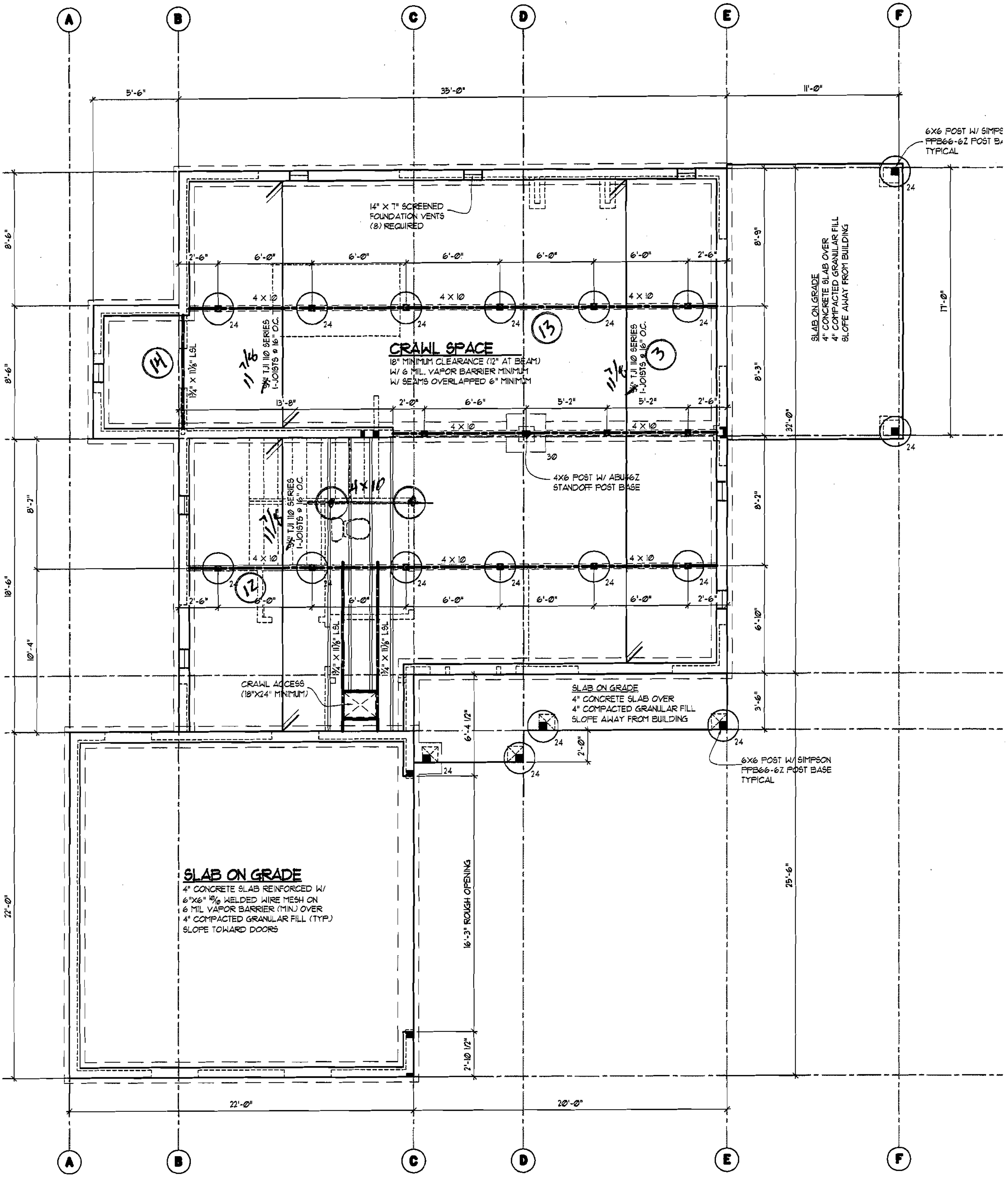
Wall Lines F:

$\frac{vf \cdot Lf - vff \cdot Lff}{49ft} = 46.53\,ft^{-1} \cdot lb$ $\quad \frac{E_f \cdot Lf - E_{ff} \cdot Lff}{49ft} = 56.73\,ft^{-1} \cdot lb$ $\quad \frac{vf \cdot Lf}{49ft} = 84.06\,ft^{-1} \cdot lb$ $\quad \frac{E_f \cdot Lf}{49ft} = 132.49\,ft^{-1} \cdot lb$



A
B
C
D
E
F
1'-0" O.H. TYP. U.N.O.
1'-0" O.H. TYP. U.N.O.
SCISSOR GABLE AT RAKED WALL
GABLE END TRUSS
5.4K worst case
4.7K worst case
3 1/2 x 7 1/2 GLB
Cont.
2
SCISSOR TRUSSES @ 24" O.C.
8:12
RIDGE
8:12
COMMON GIRDER
COMMON TRUSSES @ 24" O.C.
COMMON GIRDER
8:12
RIDGE
8:12
4:12
6:12
Worst Case 6.7K
3.75:12
5.9K Worst Case
OVER FRAMING
RIDGE
GABLED COMMON
3.75:12
OVER FRAMING
COMMON TRUSSES @ 24" O.C.
Diag
COMMON TRUSSES @ 24" O.C.
8:12
RIDGE
8:12
ATTIC ACCESS 22"x30" MINIMUM
5.4K worst case
8:12
RIDGE
8:12
4.7K
GABLE GIRDER
6:12
6"
6"
8:12
PARTIAL GABLE
RIDGE
8:12
8:12
COMMON TRUSSES @ 24" O.C.
PARTIAL GABLE
DUAL PITCH TRUSSES @ 24" O.C.
8:12
RIDGE
8:12
LINE OF PITCH CHANGE
6:12
3'-0" CANT.
1
6x8 DF#1
PARTIAL GABLE
COMMON TRUSSES @ 24" O.C.
GABLE END TRUSS
A
B
C
D
E
F


A
B
C
D
E
F
1'-0" O.H. TYP. U.N.O.
STRUCTURAL MONO GABLE
3½x7½ GLB
5½x10½ GLB
1¾" x 11⅞" LSL
9½" I-JOISTS
4:12
MONO TRUSSES @ 24" O.C.
MONO GABLE
6:12
6.7K PHD
5.9K PHD
5½x12 GLB
1¾" x 11⅞" LSL RIM BEAM
3½x7½ GLB
2'-0" CANT.
8:12
CRICKET
RIDGE
GIRDER
STRUCTURAL GABLE
MONO TRUSSES @ 24" O.C.
6x8
LSTA24
11⅞" TJI 560 SERIES I-JOISTS @ 16" O.C.
5½x15 GLB
5¼" x 11⅞" PSL
UPPER FLOOR JOISTS SHALL BE:
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
DROPPED FRAMING FOR FLUSH ENTRY SHOWERS:
PROVIDE 2X6 LEDGERS & BLOCKING AROUND
PERIMETER TO ACCEPT EDGE NAILING. SECURE
2X6 TO PERIMETER FRAMING W/ 10d COMMON
NAILS (0.148"x3") STAGGERED AT 6" O.C.



A
B
C
D
E
F
5'-6"
35'-0"
11'-0"
6X6 POST W/ SIMPS
PPB66-6Z POST B
TYPICAL
14" X 7" SCREENED
FOUNDATION VENTS
(8) REQUIRED
2'-6"
6'-0"
2'-6"
4 X 10
24
CRAWL SPACE
18" MINIMUM CLEARANCE (12" AT BEAM)
W/ 6 MIL. VAPOR BARRIER MINIMUM
W/ SEAMS OVERLAPPED 6" MINIMUM
9½" TJI 110 SERIES
I-JOISTS @ 16" O.C.
1¾" X 11⅞" LSL
13'-8"
2'-0"
6'-6"
5'-2"
30
4X6 POST W/ ABU46Z
STANDOFF POST BASE
SLAB ON GRADE
4" CONCRETE SLAB OVER
4" COMPACTED GRANULAR FILL
SLOPE AWAY FROM BUILDING
8'-6"
8'-9"
8'-3"
17'-0"
32'-0"
8'-7"
8'-2"
18'-6"
10'-4"
6'-10"
3'-6"
CRAWL ACCESS
(18"X24" MINIMUM)
6'-4 1/2"
2'-0"
6X6 POST W/ SIMPSON
PPB66-6Z POST BASE
TYPICAL
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)
SLOPE TOWARD DOORS
16'-3" ROUGH OPENING
2'-10 1/2"
22'-0"
25'-6"
20'-0"
14
13
3
12
11 7/16
11 7/8
4X10

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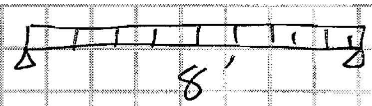
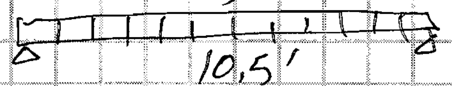
① $w_D = 15psf(28'/2) = 210plf$
$w_S = 25psf(28'/2) = 350plf$



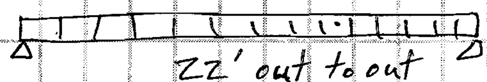
6x8 DF#1
(4x8 DF#2 @ 6' span)

② $w_D = 15psf(17'/2) = 127.5plf$
$w_S = 25psf(17'/2) = 212.5plf$



3½x7½ GLB

③ $w_D = 15psf$
$w_L = 40psf$

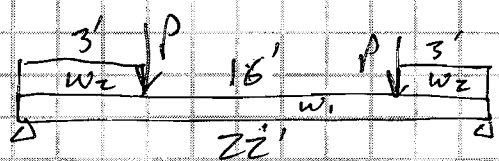


11⅞ TJI 560 @ 16" o.c.
(11⅞ TJI 110 @ 18'0" clear or less)

④ $w_{D_1} = 15plf + self$
$w_{L_1} = 40plf$

$w_{D_2} = 15psf(6'/2) = 45plf$
$w_{L_2} = 40psf(6'/2) = 120plf$

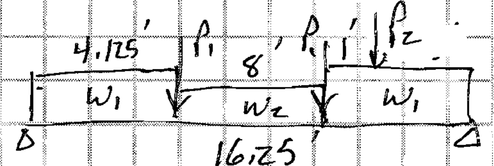
$P = 350^{\#}DL + 930^{\#}LL$



5¼x11⅞ PSL

⑤ $w_{D_1} = 15psf(24'/2 + 22'/2) + 12psf(8.5') = 447plf$
$w_{L_1} = 40psf(22'/2) = 440plf$
$w_{S_1} = 25psf(24'/2) = 300plf$

$w_{D_2} = 15psf(24'/2) + 12psf(8.5')(24/22) = 292plf$
$w_{L_2} = 40psf(26'/2) = 520plf$



5½x15 GLB

$P_1 = 840^{\#}DL + 1400^{\#}SL$ from ①

$P_2 = 650^{\#}DL + 1730^{\#}LL$ from ④

FOR RKK
JOB Chase's Lot 1

DATE 3-7-22
BY [signature]

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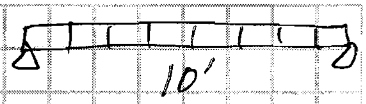
myengineer@centurytel.net

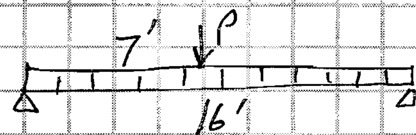
(6) $w_D = 15psf(15'/2 + 1') + 12psf(8.5') = 230plf$
$w_L = 40plf$
$w_S = 25psf(15'/2) = 187.5plf$



3½x7½ GLB

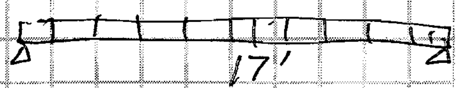
(7) $w_D = 15plf$
$w_L = 40plf$

$P = 300\#DL + 800\#LL$



3½x11⅞ LSL

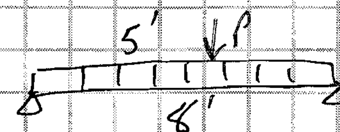
(8) $w_D = 15psf(13'/2) = 97.5plf$
$w_S = 25psf(13'/2) = 162.5plf$



5½x10½ GLB

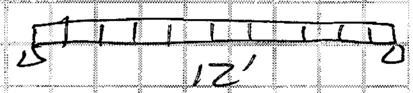
(9) $w_D = 15psf(15'/2 + 1') + 12psf(8.5') + 15psf(11'/2) = 312plf$
$w_L = 40plf$
$w_S = 25psf(15'/2 + 11'/2) = 325plf$

$P = 300\#DL + 800\#LL$



3½x7½ GLB

(10) $w_D = 15psf(5.5'/2) = 41.3plf$
$w_S = 25psf(5.5'/2) = 68.8plf$



6x8

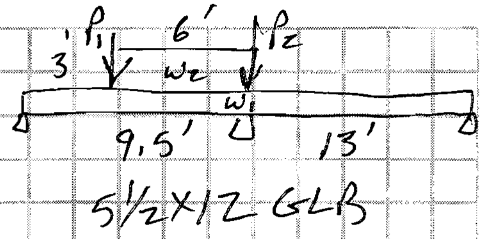
FOR RKK
JOB Chase's Lt 1

DATE 4-7-22
BY [signature]

MYERS ENGINEERING

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

(11) $w_{D_1} = 15psf(32'/2) = 240plf$
$w_{L_1} = 40psf(32'/2) = 640plf$

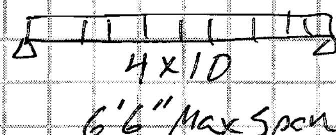


$P_1 = 2712^{\#}DL + 3988^{\#}SL$ from Girder

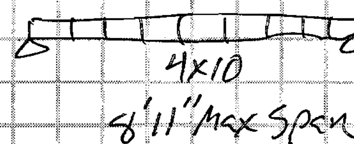
$P_2 = 2388^{\#}DL + 3512^{\#}SL$ from Girder

$w_{D_2} = 15psf(32'/2)(1.25) = 300plf$
$w_{S_2} = 25psf(32'/2)(1.25) = 500plf$

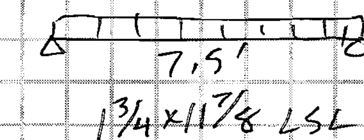
(12) $w_D = 15psf(11'/2 + 19'/2) = 225plf$
$w_L = 40psf(11'/2) + 19'/2) = 600plf$



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



(14) $w_D = 15psf(17'/2 + 2' + 5.5'/2) + 12psf(8.5' + 10') = 421plf$
$w_L = 40psf(1' + 1') = 80plf$
$w_S = 25psf(17'/2 + 5.5'/2) = 281.3plf$



FOR RKK
JOB Chase's Lt 1

DATE 4-7-22
BY MM

L/480 Live Load Deflection

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

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

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

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

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

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

How to Use These Tables

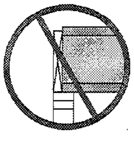
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

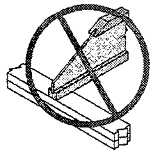
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor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

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

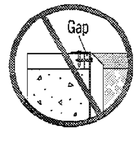
These Conditions Are NOT Permitted:



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



DO NOT bevel cut joist beyond inside face of wall.



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

Multiple Simple Beam

Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 | MYERS ENGINEERING | (c) ENERCALC INC 1983-2022

Description :

Wood Beam Design : 1. Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **6x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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

Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

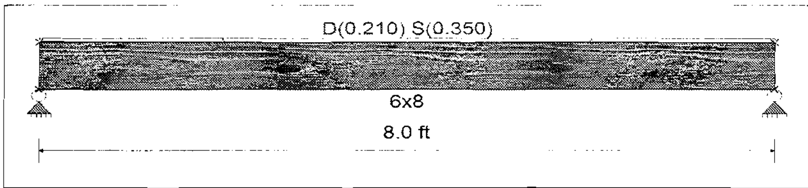
Applied Loads

Unif Load: D = 0.210, S = 0.350 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.672 : 1**
fb : Actual : 1,042.62 psi at 4.000 ft in Span # 1
Fb : Allowable : 1,552.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.353 : 1**
fv : Actual : 68.96 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.84			1.40			
Right Support	0.84			1.40			

Max Deflections

Transient Downward	0.105 in	Total Downward	0.168 in
Ratio	915	Ratio	572
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 2. Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x7.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

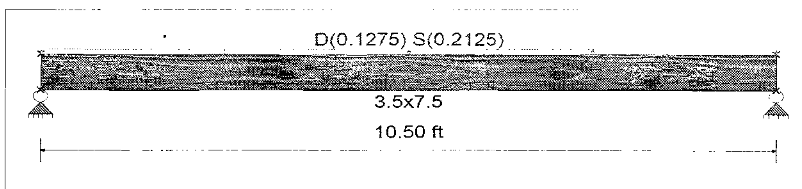
Applied Loads

Unif Load: D = 0.1275, S = 0.2125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.637 : 1**
fb : Actual : 1,713.60 psi at 5.250 ft in Span # 1
Fb : Allowable : 2,692.13 psi
Load Comb : +D+S

Max fv/FvRatio = **0.297 : 1**
fv : Actual : 90.44 psi at 9.905 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.67			1.12			
Right Support	0.67			1.12			

Max Deflections

Transient Downward	0.264 in	Total Downward	0.422 in
Ratio	477	Ratio	298
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

Wood Beam Design : 1a. Header

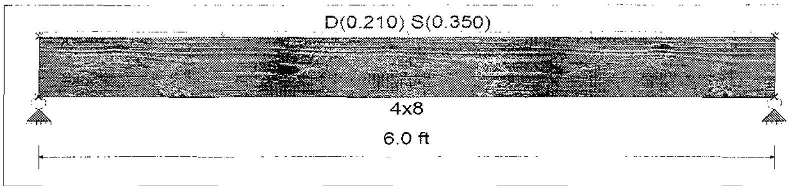
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **4x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.210, S = 0.350 k/ft, Trib= 1.0 ft



Design Summary

Max fb/Fb Ratio = **0.740** : 1
fb : Actual : 986.25 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,333.02 psi
Load Comb : +D+S

Max fv/FvRatio = **0.384** : 1
fv : Actual : 79.45 psi at 5.400 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.63			1.05			
Right Support	0.63			1.05			

Max Deflections

Transient Downward	0.058 in	Total Downward	0.092 in
Ratio	1247	Ratio	779
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 4. Beam at Showers over Garage

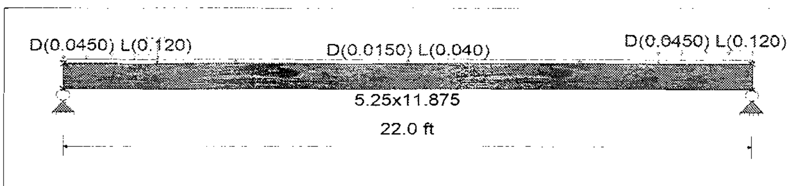
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist				Wood Grade : Parallam PSL 2.0E				
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	45.070 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
Unif Load: D = 0.0450, L = 0.120 k/ft, 0.0 to 3.0 ft, Trib= 1.0 ft
Unif Load: D = 0.0450, L = 0.120 k/ft, 19.0 to 22.0 ft, Trib= 1.0 ft
1Point: D = 0.350, L = 0.930 k @ 3.0 ft
2Point: D = 0.350, L = 0.930 k @ 19.0 ft



Design Summary

Max fb/Fb Ratio = **0.265** : 1
fb : Actual : 769.28 psi at 11.000 ft in Span # 1
Fb : Allowable : 2,900.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.180** : 1
fv : Actual : 52.22 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.65	1.73					
Right Support	0.65	1.73					

Max Deflections

Transient Downward	0.378 in	Total Downward	0.520 in
Ratio	698	Ratio	507
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 MYERS ENGINEERING

Wood Beam Design : 5. Garage Door Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **5.5x15, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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

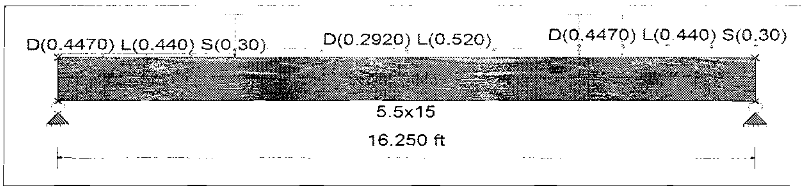
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.4470, L = 0.440, S = 0.30 k/ft, 0.0 ft to 4.125 ft, Trib= 1.0 ft
Unif Load: D = 0.2920, L = 0.520 k/ft, 4.125 to 12.125 ft, Trib= 1.0 ft
Unif Load: D = 0.4470, L = 0.440, S = 0.30 k/ft, 12.125 to 16.250 ft, Trib= 1.0 ft
1Point: D = 0.840, S = 1.40 k @ 4.125 ft
2Point: D = 0.840, S = 1.40 k @ 12.125 ft
3Point: D = 0.650, L = 1.730 k @ 13.125 ft

Design Summary

Max fb/Fb Ratio = **0.867** : 1
fb : Actual : 2,021.99 psi at 8.667 ft in Span # 1
Fb : Allowable : 2,332.08 psi
Load Comb : +D+L
Max fv/FvRatio = **0.588** : 1
fv : Actual : 155.71 psi at 15.004 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.98	4.23		2.64			
Right Support	4.38	5.29		2.64			

Max Deflections

Transient Downward	0.334 in	Total Downward	0.646 in
Ratio	583	Ratio	301
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 6. Header at Dining Rm

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x7.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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

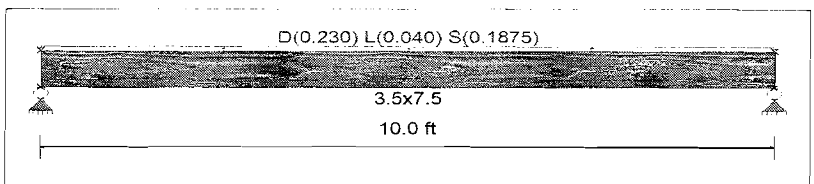
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.230, L = 0.040, S = 0.1875 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.708** : 1
fb : Actual : 1,908.57 psi at 5.000 ft in Span # 1
Fb : Allowable : 2,696.62 psi
Load Comb : +D+S
Max fv/FvRatio = **0.344** : 1
fv : Actual : 104.97 psi at 9.400 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.15	0.20		0.94			
Right Support	1.15	0.20		0.94			

Max Deflections

Transient Downward	0.191 in	Total Downward	0.426 in
Ratio	626	Ratio	281
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 7. Beam at Master Shower

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x11.875, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

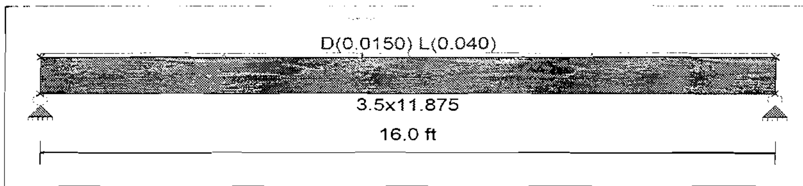
Wood Species : iLevel Truss Joist   Wood Grade : TimberStrand LSL 1.55E

Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	45.010 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
1Point: D = 0.30, L = 0.80 k @ 7.0 ft

Design Summary



Max fb/Fb Ratio = **0.380** : 1
fb : Actual : 883.27 psi at 6.987 ft in Span # 1
Fb : Allowable : 2,325.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.117** : 1
fv : Actual : 36.30 psi at 0.000 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.29	0.77					
Right Support	0.25	0.67					

Max Deflections

Transient Downward	0.232 in	Total Downward	0.319 in
Ratio	828	Ratio	602
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 8. Roof Beam at Covered Patio

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **5.5x10.5, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

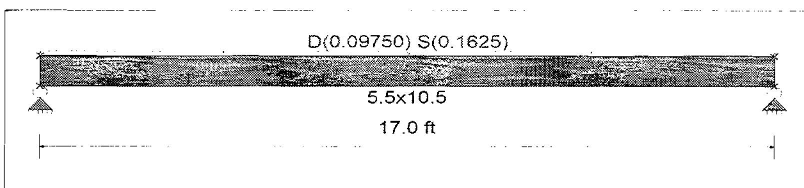
Wood Species : DF/DF   Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.09750, S = 0.1625 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.404** : 1
fb : Actual : 1,115.25 psi at 8.500 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.170** : 1
fv : Actual : 51.66 psi at 16.150 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.83			1.38			
Right Support	0.83			1.38			

Max Deflections

Transient Downward	0.321 in	Total Downward	0.514 in
Ratio	634	Ratio	396
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 9. Header at SGD

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x7.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

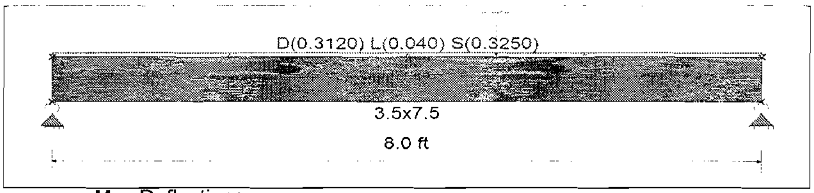
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.3120, L = 0.040, S = 0.3250 k/ft, Trib= 1.0 ft
1Point: D = 0.30, L = 0.80 k @ 5.0 ft

Design Summary



Max fb/Fb Ratio = **0.827 : 1**
fb : Actual : 2,243.00 psi at 4.587 ft in Span # 1
Fb : Allowable : 2,712.18 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.477 : 1**
fv : Actual : 145.50 psi at 7.387 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.36	0.46		1.30			
Right Support	1.44	0.66		1.30			

Max Deflections

Transient Downward	0.136 in	Total Downward	0.314 in
Ratio	706	Ratio	305
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 10. Roof Beam at Front Porch

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **6x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

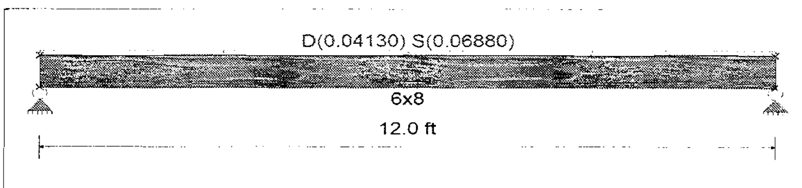
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	875.0 psi	Fc - Prll	600.0 psi	Fv	170.0 psi	Ebend- xx	1,300.0 ksi	Density	31.210 pcf
Fb - Compr	875.0 psi	Fc - Perp	625.0 psi	Ft	425.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.04130, S = 0.06880 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.458 : 1**
fb : Actual : 461.22 psi at 6.000 ft in Span # 1
Fb : Allowable : 1,006.25 psi
Load Comb : +D+S

Max fv/FvRatio = **0.111 : 1**
fv : Actual : 21.62 psi at 11.400 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.25			0.41			
Right Support	0.25			0.41			

Max Deflections

Transient Downward	0.128 in	Total Downward	0.205 in
Ratio	1121	Ratio	700
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 12. Crawl Beam at bearing wall

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

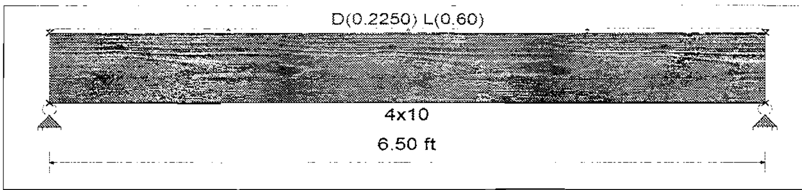
BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.2250, L = 0.60 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.981** : 1
fb : Actual : 1,047.54 psi at 3.250 ft in Span # 1
Fb : Allowable : 1,068.31 psi
Load Comb : +D+L
Max fv/FvRatio = **0.529** : 1
fv : Actual : 95.24 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.73	1.95					
Right Support	0.73	1.95					

Max Deflections

Transient Downward	0.066 in	Total Downward	0.090 in
Ratio	1189	Ratio	864
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 13. Crawl Beam NOT at bearing wall

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

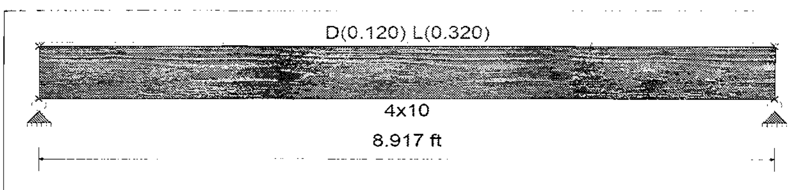
BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.120, L = 0.320 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.988** : 1
fb : Actual : 1,051.43 psi at 4.459 ft in Span # 1
Fb : Allowable : 1,063.81 psi
Load Comb : +D+L
Max fv/FvRatio = **0.421** : 1
fv : Actual : 75.74 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.54	1.43					
Right Support	0.54	1.43					

Max Deflections

Transient Downward	0.124 in	Total Downward	0.170 in
Ratio	863	Ratio	628
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: 11. Beam at Great Rm/Dining Rm

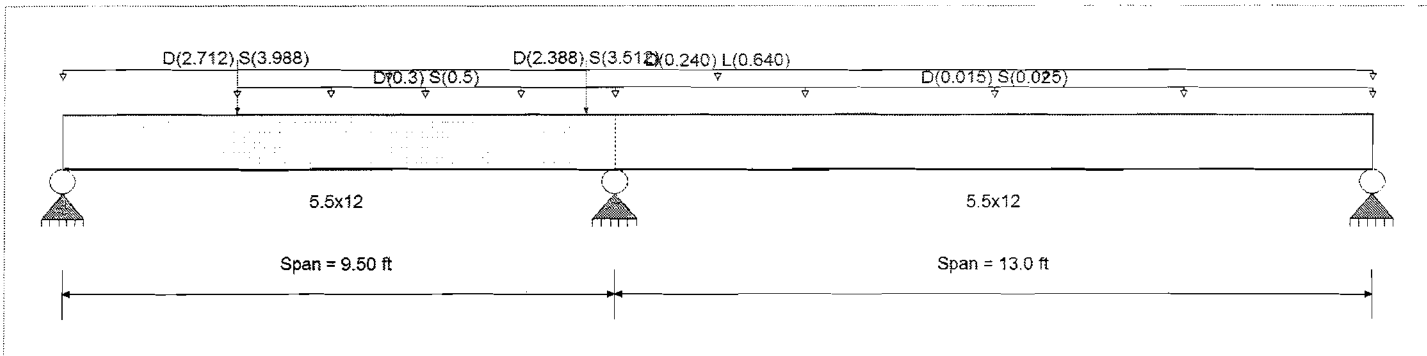
CODE REFERENCES

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.

Beam self weight NOT internally calculated and added
Loads on all spans...
 Uniform Load on ALL spans : D = 0.240, L = 0.640 k/ft
Load for Span Number 1
 Uniform Load : D = 0.30, S = 0.50 k/ft, Extent = 3.0 -->> 9.50 ft, Tributary Width = 1.0 ft
 Point Load : D = 2.712, S = 3.988 k @ 3.0 ft
 Point Load : D = 2.388, S = 3.512 k @ 9.0 ft
Load for Span Number 2
 Uniform Load : D = 0.0150, S = 0.0250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.899 : 1**	Maximum Shear Stress Ratio	=	**0.638 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
fb: Actual	=	1,662.28 psi	fv: Actual	=	194.53 psi
Fb: Allowable	=	1,850.00 psi	Fv: Allowable	=	304.75 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	9.500 ft	Location of maximum on span	=	8.545 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.153 in	Ratio =	1022 >=360	Span: 2 : L Only
Max Upward Transient Deflection	-0.058 in	Ratio =	2677 >=360	Span: 2 : S Only
Max Downward Total Deflection	0.168 in	Ratio =	680 >=240	Span: 2 : +D+L
Max Upward Total Deflection	-0.053 in	Ratio =	2949 >=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 = 9.50 ft	1	0.334	0.326	0.90	1.000	1.00	1.00	1.00	1.00	1.00	7.93	720.86	2160.00	3.42	77.71	238.50
Length = 13.0 ft	2	0.405	0.326	0.90	1.000	1.00	1.00	1.00	1.00	1.00	7.43	675.01	1665.00	1.99	77.71	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 = 9.50 ft	1	0.899	0.600	1.00	1.000	1.00	1.00	1.00	1.00	1.00	18.29	1,662.28	1850.00	6.99	158.89	265.00
Length = 13.0 ft	2	0.899	0.600	1.00	1.000	1.00	1.00	1.00	1.00	1.00	18.29	1,662.28	1850.00	6.38	158.89	265.00

DESCRIPTION: 11. Beam at 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
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.604	0.500	1.15	1.000	1.00	1.00	1.00	1.00	1.00	18.35	1,668.41	2760.00	6.70	152.30	304.75
Length = 13.0 ft	2	0.539	0.500	1.15	1.000	1.00	1.00	1.00	1.00	1.00	12.62	1,147.12	2127.50	2.53	152.30	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 = 9.50 ft	1	0.612	0.418	1.25	1.000	1.00	1.00	1.00	1.00	1.00	15.57	1,415.46	2312.50	6.10	138.59	331.25
Length = 13.0 ft	2	0.612	0.418	1.25	1.000	1.00	1.00	1.00	1.00	1.00	15.57	1,415.46	2312.50	5.28	138.59	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 = 9.50 ft	1	0.832	0.638	1.15	1.000	1.00	1.00	1.00	1.00	1.00	19.46	1,769.55	2127.50	8.56	194.53	304.75
Length = 13.0 ft	2	0.832	0.638	1.15	1.000	1.00	1.00	1.00	1.00	1.00	19.46	1,769.55	2127.50	5.69	194.53	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 = 9.50 ft	1	0.214	0.209	1.60	1.000	1.00	1.00	1.00	1.00	1.00	9.04	821.78	3840.00	3.90	88.59	424.00
Length = 13.0 ft	2	0.260	0.209	1.60	1.000	1.00	1.00	1.00	1.00	1.00	8.46	769.51	2960.00	2.27	88.59	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 = 9.50 ft	1	0.622	0.478	1.60	1.000	1.00	1.00	1.00	1.00	1.00	20.24	1,840.42	2960.00	8.92	202.69	424.00
Length = 13.0 ft	2	0.622	0.478	1.60	1.000	1.00	1.00	1.00	1.00	1.00	20.24	1,840.42	2960.00	5.89	202.69	424.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.113	0.110	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.76	432.52	3840.00	2.05	46.63	424.00
Length = 13.0 ft	2	0.137	0.110	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.46	405.01	2960.00	1.19	46.63	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 = 9.50 ft	1	0.086	0.084	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.65	331.60	3840.00	1.57	35.75	424.00
Length = 13.0 ft	2	0.105	0.084	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.42	310.50	2960.00	0.91	35.75	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.1675	4.299		0.0000	0.000
+D+L	2	0.1753	7.480	+D+S	-0.0437	1.888

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	7.038	21.311	4.411
Overall MINimum	3.479	7.833	-0.237
D Only	3.007	8.552	1.086
+D+L	4.904	17.730	4.411
+D+S	6.485	16.385	0.849
+D+0.750L	4.429	15.436	3.580
+D+0.750L+0.750S	7.038	21.311	3.402
+0.60D	1.804	5.131	0.652
L Only	1.897	9.179	3.325
S Only	3.479	7.833	-0.237

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

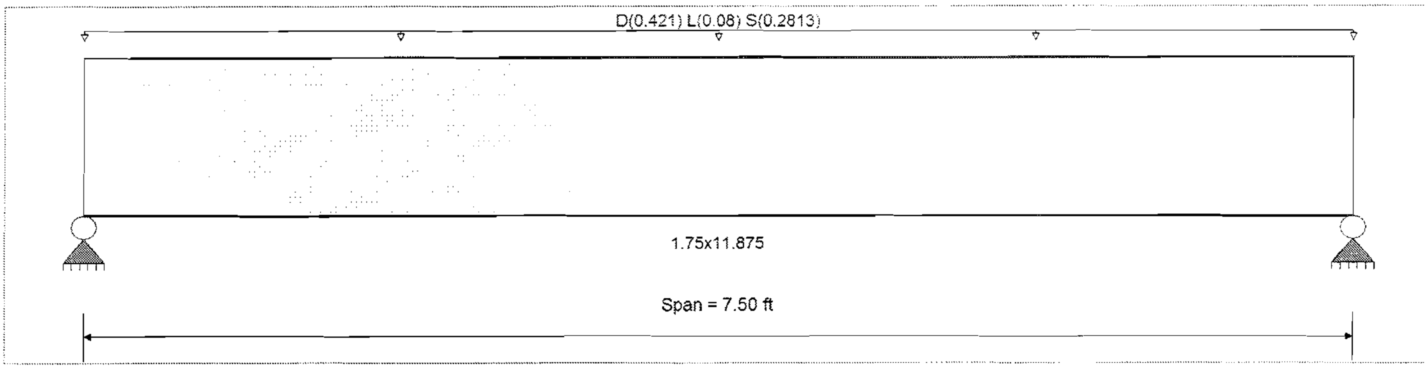
DESCRIPTION: 14. Floor beam at Grid B/Pantry

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 +	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.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.4210, L = 0.080, S = 0.2813 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.539 : 1	Maximum Shear Stress Ratio	=	0.393 : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
fb: Actual	=	1,440.73 psi	fv: Actual	=	140.14 psi
Fb: Allowable	=	2,673.75 psi	Fv: Allowable	=	356.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.750 ft	Location of maximum on span	=	6.515 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.053 in	Ratio =	1691 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.133 in	Ratio =	677 >=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 = 7.50 ft	1	0.413	0.301	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.96	863.66	2092.50	1.16	84.01	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 = 7.50 ft	1	0.442	0.322	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.52	1,027.77	2325.00	1.39	99.97	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 = 7.50 ft	1	0.539	0.393	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.94	1,440.73	2673.75	1.94	140.14	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 = 7.50 ft	1	0.340	0.248	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.38	986.74	2906.25	1.33	95.98	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 = 7.50 ft	1	0.531	0.387	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.87	1,419.55	2673.75	1.91	138.08	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 = 7.50 ft	1	0.265	0.193	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.37	984.57	3720.00	1.33	95.77	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

Project File: chases lot 1.ec6

LIC# : KW-06015659, Build:20.22.3.31 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: 14. Floor beam at Grid B/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 = 7.50 ft	1	0.406	0.296	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.18	1,510.23	3720.00	2.04	146.90	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 = 7.50 ft	1	0.139	0.102	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.78	518.19	3720.00	0.70	50.41	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 = 7.50 ft	1	0.107	0.078	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.36	397.28	3720.00	0.54	38.64	496.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.1329	3.777		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.634	2.634
Overall MINimum	1.055	1.055
D Only	1.579	1.579
+D+L	1.879	1.879
+D+S	2.634	2.634
+D+0.750L	1.804	1.804
+D+0.750L+0.750S	2.595	2.595
+0.60D	0.947	0.947
L Only	0.300	0.300
S Only	1.055	1.055

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)

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PROJECT : **Chase's Corner - Lot 1**

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Maximum Load For 3-2x4 HF Stud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 10 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

Maximum Load For 2-2x4 HFStud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 10 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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Maximum Load For 4x4 HF#2 Treated Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 6.25 \cdot ft$

$F_c := 1040 \cdot psi$ $C_D := 1$ $C_{Fb} := 1$ $C_M := 1$ $C_t := 1$ $C_L := 1$ $C_{Fc} := 1$

$E' := 1235000 \cdot psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc}$ $F''_c = 1040 \cdot psi$

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

4x4 Treated Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

$A := t \cdot h$ $A = 12.2 \cdot in^2$

$I := \frac{t \cdot h^3}{12}$ $I = 12.5 \cdot in^4$

$S := \frac{I \cdot 2}{h}$ $S = 7.1 \cdot in^3$

$C_p = 0.6$

$F'_c := C_p \cdot F''_c$ $F'_c = 622 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 7618 \cdot lb$ (Maximum post Capacity)