MYERS ENGINEERING

Structural Calculations

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Date: 2022.06.15 12:26:03 -07'00'

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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: SFR for RKK Construction
3419 72nd Place Southeast
Mercer Island, WA

June 15, 2022

2018 INTERNATIONAL BUILDING CODE
100 MPH BASIC WIND, EXPOSURE C, K_{zt} = 1.60
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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PROJECT : **3419 72nd Place SE**

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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

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

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.

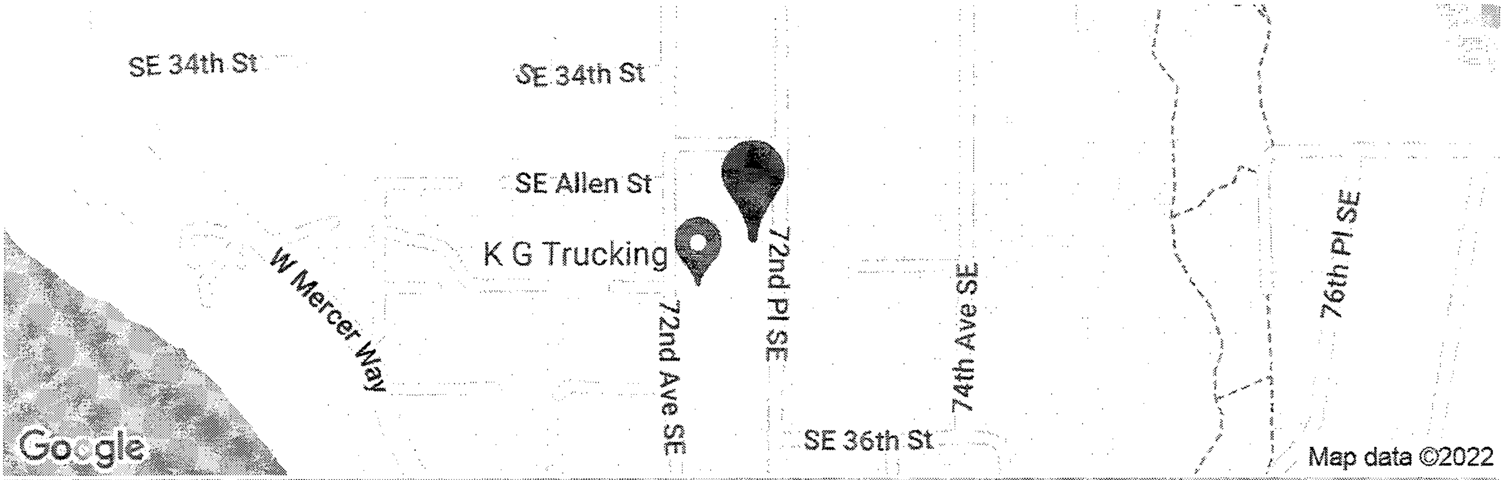
ENGINEERED I-JOISTS

-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

3419 72nd Place SE

Latitude, Longitude: 47.5794, -122.2424



Date	5/27/2022, 1:01:54 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.412	MCE_R ground motion. (for 0.2 second period)
S_1	0.491	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.694	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8 0.884	Site-modified spectral acceleration value
S_{DS}	1.129	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	0.604	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.725	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.412	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.565	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.424	Factored deterministic acceleration value. (0.2 second)
S1RT	0.491	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.548	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.381	Factored deterministic acceleration value. (1.0 second)
PGAd	1.174	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	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)

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

$S_s := 1.412$ $S_1 := 0.491$ $S_{ms} := 1.694$ $S_{m1} := 0.884$

Equation 11.4-3 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.13$ Equation 11.4-4 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.59$

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

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

Plan Area for Each Level:

$A_1 := 1710ft^2 \cdot S_a$ (Upper Roof) $A_{2a} := 1505ft^2$ (Upper Floor) $A_{2b} := 360ft^2 \cdot S_a$ (Lower Roof) $A_3 := 700ft^2$ (Main Floor Tributary)

Plan Perimeter for Each Level:

$P_1 := 2(34ft) + 2(53ft)$ (Upper Floor) $P_2 := 2(34ft) + 2(53ft)$ (Main Floor) $P_3 := 2(12ft) + (32ft)$ (Lower level Tributary)

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

Weight of Structure at Each Level:

Story Weight at Upper Floor:

$$w_1 := 15 \cdot psf \cdot A_1 + 12 \cdot psf \cdot 4.5 \cdot ft \cdot P_1 = 40223.46\,lb$$

Story Weight at Main Floor:

$$w_2 := 15 \cdot psf \cdot (A_{2a} + A_{2b}) + 12 \cdot psf \cdot (4 \cdot ft \cdot P_1 + 5ft \cdot P_2) = 47856.99\,lb$$

Story Weight at Lower Level:

$$w_3 := 15 \cdot psf \cdot (A_3) + 12 \cdot psf \cdot (5 \cdot ft \cdot P_2 + 4ft \cdot P_3) = 23628\,lb$$

$$W := w_1 + w_2 + w_3 = 111708.46\,lb$$

CRAWL SPACE # I VENTILATION

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

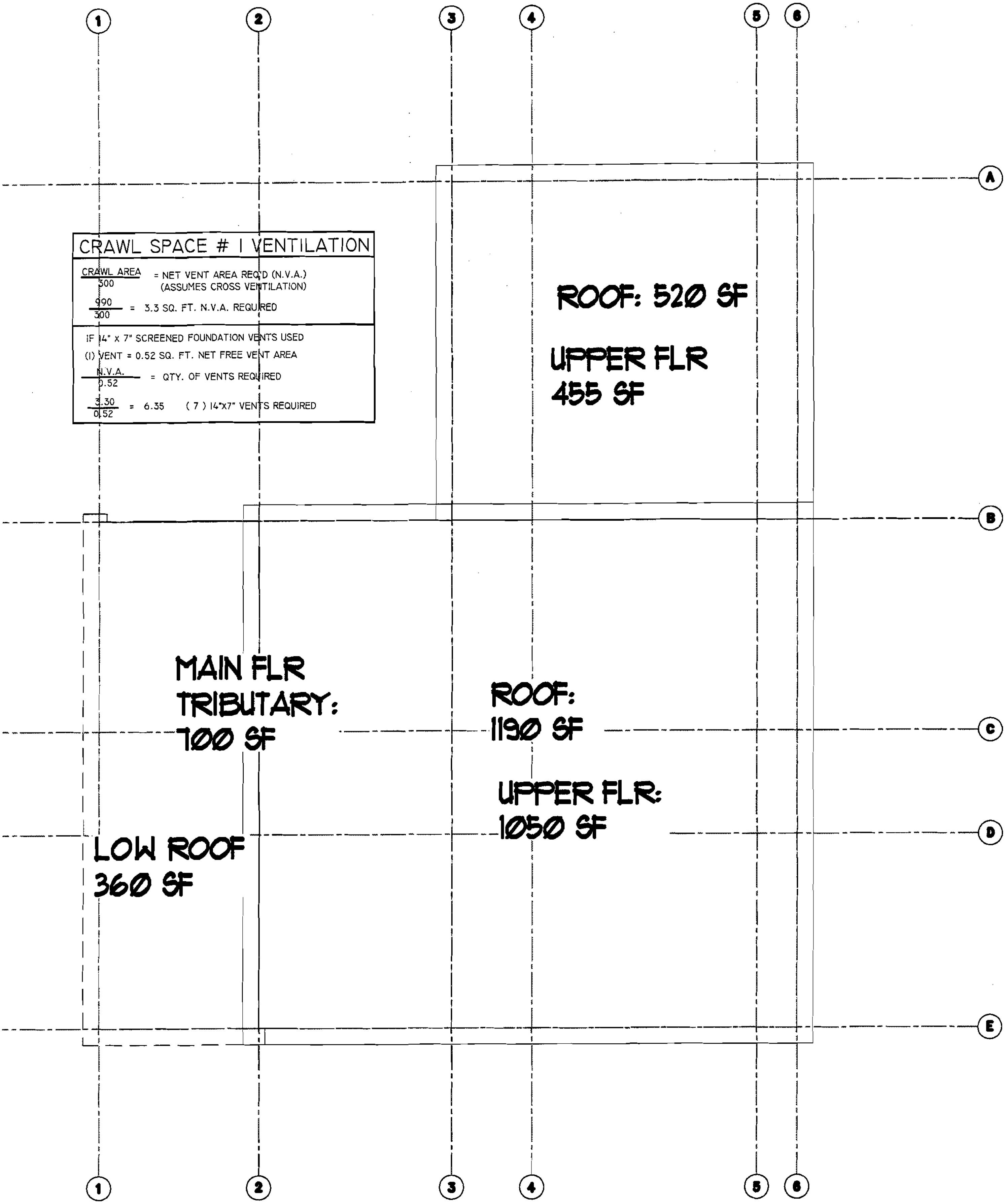
$\frac{990}{300}$ = 3.3 SQ. FT. N.V.A. REQUIRED

IF 14" x 7" SCREENED FOUNDATION VENTS USED

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

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

$\frac{3.30}{0.52}$ = 6.35 (7) 14"x7" VENTS REQUIRED



Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $h_n := 25$ (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: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.41$ (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: $V_E := C_s \cdot W = 19408.63\,\text{lb}$

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: $k := 1$

$h_1 := 26\text{ft}$ $h_2 := 18\text{ft}$ $h_3 := 7\text{ft}$ (Height from base to level x)

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2 + w_3 \cdot h_3)} = 0.5$$ $F_1 := C_{v1} \cdot V_E = 9793.22\,\text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2 + w_3 \cdot h_3)} = 0.42$$ $F_2 := C_{v2} \cdot V_E = 8066.6\,\text{lb}$ Story Shear at Main Floor

$$C_{v3} := \frac{(w_3 \cdot h_3)}{(w_1 \cdot h_1 + w_2 \cdot h_2 + w_3 \cdot h_3)} = 0.08$$ $F_3 := C_{v3} \cdot V_E = 1548.81\,\text{lb}$ Story Shear at Lower Level

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WIND DESIGN

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

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

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 25 \cdot ft$ Mean Roof Height as per Sect. 26.2

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

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

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

$x := 61ft$ $H := 300ft$ $L_h := 840ft$ $z := h$ $\gamma := 3$ $\mu := 1.5$

$K_1 := 0.85\left(\frac{H}{L_h}\right) = 0.3$ $K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.95$ $K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.91$ $K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1.6$

Matches City Map

$G := 0.85$ 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):

$z_g := 900ft$ $\alpha := 9.5$ (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)

$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.89$ $K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.85$ $K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.95$

External Pressure Coefficients w/ Roof Pitch = 5/12 (22 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 34ft$ $B_{fb} := 53ft$ $\frac{L_{fb}}{B_{fb}} = 0.64$ $\frac{h}{L_{fb}} = 0.74$

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

$C_{pf2} := 0.01$ Windward Roof

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

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

Side to Side:

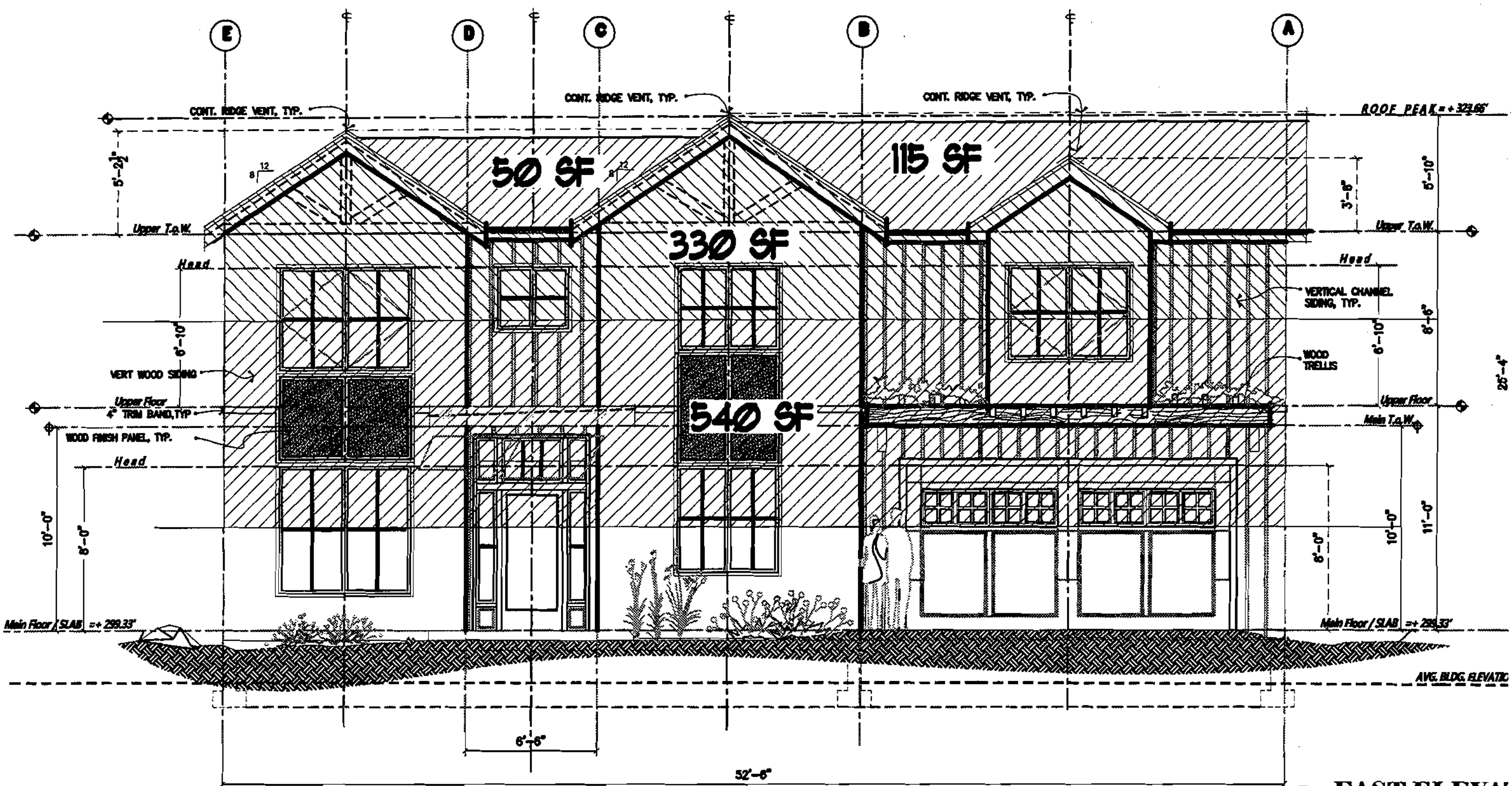
$L_{ss} := 53ft$ $B_{ss} := 34ft$ $\frac{L_{ss}}{B_{ss}} = 1.56$ $\frac{h}{L_{ss}} = 0.47$

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

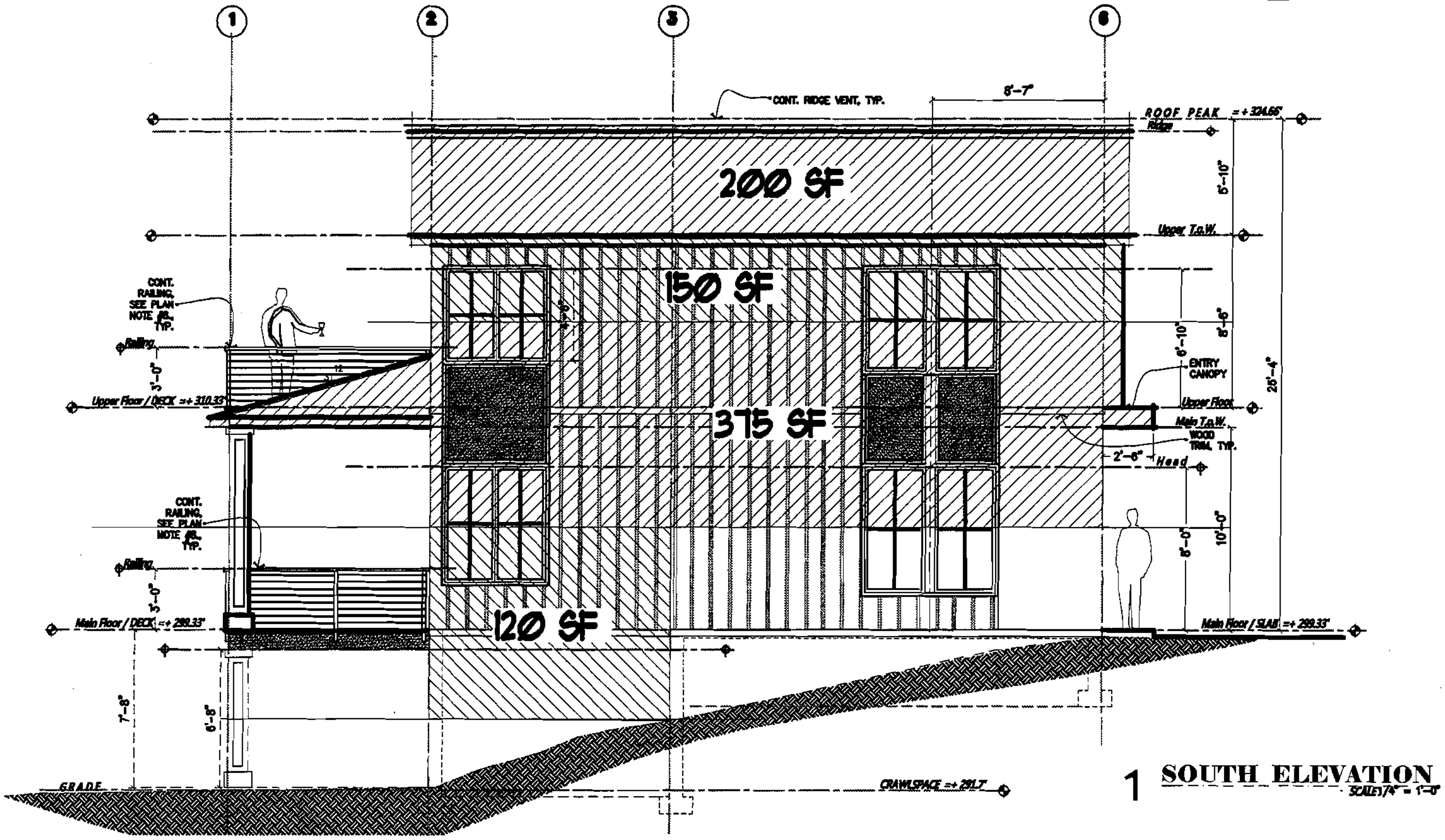
$C_{ps2} := 0.3$ Windward Roof

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

$C_{ps4} := -.4$ Leeward Wall



2 EAST ELEVA



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

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 31.03$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 29.52$ $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 32.87$

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 $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 21.1\,lb \cdot ft^{-2}$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 20.07\,lb \cdot ft^{-2}$

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 0.28\,lb \cdot ft^{-2}$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -16.77\,lb \cdot ft^{-2}$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -13.97\,lb \cdot ft^{-2}$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 8.38\,lb \cdot ft^{-2}$

Leeward Roof Side to Side $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -16.77\,lb \cdot ft^{-2}$

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -11.18\,lb \cdot ft^{-2}$

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:

$p_{wr1} - p_{lr1} = 17.04\,lb \cdot ft^{-2}$ $p_{ww1} - p_{lw1} = 35.07\,lb \cdot ft^{-2}$ $p_{ww2} - p_{lw1} = 34.05\,lb \cdot ft^{-2}$

$p_{wr2} - p_{lr2} = 25.15\,lb \cdot ft^{-2}$ $p_{ww1} - p_{lw2} = 32.28\,lb \cdot ft^{-2}$ $p_{ww2} - p_{lw2} = 31.25\,lb \cdot ft^{-2}$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})165ft^2 + (p_{ww1} - p_{lw1}) \cdot 330 \cdot ft^2 = 14385.54\,lb$$

Wind Pressure at Main Floor (Front to Back):

$$V_{2W} := (p_{wr1} - p_{lr1})0ft^2 + (p_{ww2} - p_{lw1}) \cdot 540 \cdot ft^2 = 18384.75\,lb$$

Wind Pressure at Upper Roof (Side to Side):

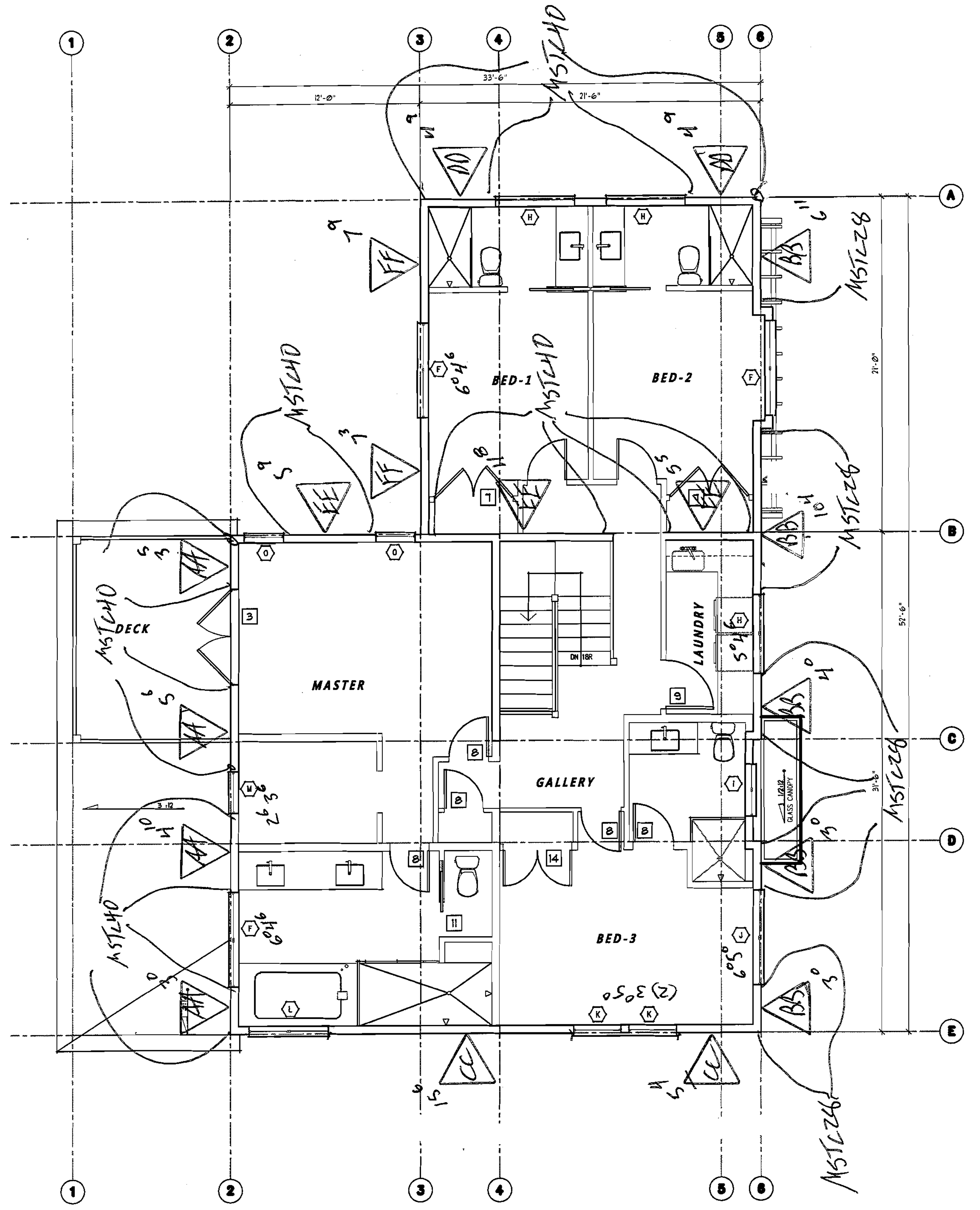
$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 200ft^2 + (p_{ww1} - p_{lw2}) \cdot 150ft^2 = 9871.01\,lb$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 375ft^2 = 11719.35\,lb$$

Wind Pressure at Lower Level (Side to Side):

$$V_{5W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 120ft^2 = 3750.19\,lb$$



33'-6"
12'-0"
21'-6"
21'-0"
52'-6"
31'-6"
BED-1
BED-2
DECK
MASTER
LAUNDRY
DN 18R
GALLERY
BED-3
GLASS CANOPY
MSTC40
MSTC26

WALL AA:

Story Shear due to Wind: $V_{3W} = 9871.01\,lb$ Story Shear due to Seismic: $F_1 = 9793.22\,lb$

Bldg Width in direction of Load: $L_t := 33.5\cdot ft$ Distance between shear walls: $L_1 := 33.5\cdot ft$

Shear Wall Length: $Laa := \left[3\left(\frac{6}{8.5}\right) + 4.83 + 5.5 + 3.42\left(\frac{6.83}{8.5}\right)\right]ft = 15.2\,ft$

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

Wind Force: $vaa := \dfrac{\frac{0.6V_{3W}}{L_t}\cdot\frac{L_1}{2}}{Laa}$ Seismic Force: $\rho := 1.0$ $E_{aa} := \dfrac{0.7\rho\cdot\frac{0.67F_1}{L_t}\cdot\frac{L_1}{2}}{Laa}$

$vaa = 194.88\,lb\cdot ft^{-1}$ $\frac{vaa}{C_o} = 194.88\,lb\cdot ft^{-1}$ $E_{aa} = 151.13\,lb\cdot ft^{-1}$ $\frac{E_{aa}}{C_o} = 151.13\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{aa} := 3\cdot ft$ Plate Height: $Pt := 8.5\cdot ft$

$W_{aa} := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 172.5\,lb$

Chord Force:

$CFaa_w := \frac{vaa\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 1656.46\,lb$ $CFaa_s := \frac{E_{aa}\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 1284.59\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = 1552.96\,lb$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 1208.37\,lb$

Simpson MSTC40 strap

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

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

$B_p := \frac{(Z_N\cdot C_D\cdot C_o)}{vaa} = 0.84\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{aa}} = 1.08\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vaa} = 7.06\,ft$ $\frac{(Z_B\cdot C_o)}{E_{aa}} = 9.1\,ft$

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

WALL BB:

Story Shear due to Wind: $V_{3W} = 9871.01\,lb$

Story Shear due to Seismic: $F_1 = 9793.22\,lb$

Bldg Width in direction of Load: $L_t := 33.5\cdot ft$

Distance between shear walls: $L_1 := 33.5\cdot ft$

Shear Wall Length: $Lbb := \left[2\cdot 3\left(\frac{6}{8.5}\right) + 4\left(\frac{8}{8.5}\right) + 10.33 + 6.92\right] ft = 25.25\,ft$

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

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

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

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

$vbb = 117.28\,lb\cdot ft^{-1}$ $\quad \frac{vbb}{C_o} = 117.28\,lb\cdot ft^{-1}$

$E_{bb} = 135.75\,lb\cdot ft^{-1}$ $\quad \frac{E_{bb}}{C_o} = 135.75\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{bb} := 3\cdot ft$ $\quad$ Plate Height: $Pt := 8\cdot ft$

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

$DLRbb := \frac{W_{bb}\cdot L_{bb}}{2}$ $\quad DLRbb = 165\,lb$

Chord Force:

$CFbb_w := \frac{vbb\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}}$ $\quad CFbb_w = 938.23\,lb$

$CFbb_s := \frac{E_{bb}\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}}$ $\quad CFbb_s = 1085.98\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6\cdot DLRbb = 839.23\,lb$

$HDFbb_s := CFbb_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRbb = 1013.07\,lb$

Simpson MSTC28 strap

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{vbb} = 1.39\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_{bb}} = 1.2\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vbb} = 11.73\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_{bb}} = 10.14\,ft$

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

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PROJECT : **3419 72nd Place SE**

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WALL CC:

Story Shear due to Wind: $V_{1W} = 14385.54\,lb$ Story Shear due to Seismic: $F_1 = 9793.22\,lb$

Bldg Width in direction of Load: $L_t := 52.5\cdot ft$ Distance between shear walls: $L_1 := 31.5\cdot ft$

Shear Wall Length: $Lcc := (5.33 + 15.5)ft = 20.83\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 0.85$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vcc := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1}{2}}{Lcc}$ Seismic Force: $\rho := 1.0$ $E_{cc} := \dfrac{0.7\rho\cdot\frac{(0.67F_1)}{2}}{Lcc}$

$vcc = 124.31\,lb\cdot ft^{-1}$ $\frac{vcc}{C_o} = 146.25\,lb\cdot ft^{-1}$ $E_{cc} = 110.25\,lb\cdot ft^{-1}$ $\frac{E_{cc}}{C_o} = 129.71\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{cc} := 5.33\cdot ft$ Plate Height: $Pt := 8.5\cdot ft$

$W_{cc} := (15\cdot psf)\cdot 7\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRcc := \frac{W_{cc}\cdot L_{cc}}{2}$ $DLRcc = 506.35\,lb$

Chord Force:

$CFcc_w := \frac{vcc\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_w = 1243.11\,lb$ $CFcc_s := \frac{E_{cc}\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_s = 1102.5\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 939.3\,lb$ $HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS})\cdot DLRcc = 878.75\,lb$

No Holdown Required

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

$Z_N := 102\cdot lb$ $C_D := 1.6$
$B_N := \frac{(C_D\cdot Z_N\cdot C_o)}{vcc} = 1.12\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{cc}} = 1.26\,ft$

16d @ 8" o.c.

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

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$
$As := \frac{(Z_B\cdot C_o)}{vcc} = 9.41\,ft$ $\frac{(Z_B\cdot C_o)}{E_{cc}} = 10.61\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 14385.54\,lb$ Story Shear due to Seismic: $F_1 = 9793.22\,lb$

Bldg Width in direction of Load: $L_t := 52.5 \cdot ft$ Distance between shear walls: $L_1 := 21 \cdot ft$

Shear Wall Length: $Ldd := (2 \cdot 4.75)ft = 9.5\,ft$

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: $vdd := \frac{\frac{0.6V_{1W}}{L_t} \cdot \frac{L_1}{2}}{Ldd}$ Seismic Force: $\rho := 1.0$ $E_{dd} := \frac{0.7\rho \cdot \frac{0.33F_1}{2}}{Ldd}$

$vdd = 181.71\,lb \cdot ft^{-1}$ $\frac{vdd}{C_o} = 181.71\,lb \cdot ft^{-1}$ $E_{dd} = 119.06\,lb \cdot ft^{-1}$ $\frac{E_{dd}}{C_o} = 119.06\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{dd} := 4.75 \cdot ft$ Plate Height: $Pt := 8.5 \cdot ft$

$W_{dd} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRdd := \frac{W_{dd} \cdot L_{dd}}{2}$ $DLRdd = 273.12\,lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_w = 1544.55\,lb$ $CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $CFdd_s = 1012.05\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 1380.68\,lb$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 891.36\,lb$

Simpson MSTC40 strap

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 0.9\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 1.37\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vdd} = 7.57\,ft$ $\frac{(Z_B \cdot C_o)}{E_{dd}} = 11.56\,ft$

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

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WALL EE:

Story Shear due to Wind: $V_{1W} = 14385.54\,\mathrm{lb}$ Story Shear due to Seismic: $F_1 = 9793.22\,\mathrm{lb}$

Bldg Width in direction of Load: $L_t := 52.5\cdot\mathrm{ft}$ Distance between shear walls: $L_1 := 31.5\cdot\mathrm{ft}$ $L_2 := 21\mathrm{ft}$

Shear Wall Length:

$$Lee := (5.42 + 11.67 + 5.75)\mathrm{ft} = 22.84\,\mathrm{ft}$$

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

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

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

$vee = 188.95\,\mathrm{lb}\cdot\mathrm{ft}^{-1}$ $\frac{vee}{C_o} = 188.95\,\mathrm{lb}\cdot\mathrm{ft}^{-1}$ $E_{ee} = 150.07\,\mathrm{lb}\cdot\mathrm{ft}^{-1}$ $\frac{E_{ee}}{C_o} = 150.07\,\mathrm{lb}\cdot\mathrm{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{ee} := 5.42\mathrm{ft}$ Plate Height: $Pt := 8.5\cdot\mathrm{ft}$

$W_{ee} := (15\cdot\mathrm{psf})\cdot 7\cdot\mathrm{ft} + (10\cdot\mathrm{psf})\cdot Pt + (10\mathrm{psf})\cdot 0\mathrm{ft}$

$DLRee := \frac{W_{ee}\cdot L_{ee}}{2}$ $DLRee = 514.9\,\mathrm{lb}$

Chord Force:

$CFee_w := \frac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_w = 1606.09\,\mathrm{lb}$ $CFee_s := \frac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_s = 1275.61\,\mathrm{lb}$

Holdown Force:

$HDFee_w := CFee_w - 0.6\cdot DLRee = 1297.15\,\mathrm{lb}$ $HDFee_s := CFee_s - \left(0.6 - 0.14S_{DS}\right)DLRee = 1048.07\,\mathrm{lb}$

Simpson MSTC40

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

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

$B_p := \frac{\left(Z_N\cdot C_D\cdot C_o\right)}{vee} = 0.86\,\mathrm{ft}$ $\frac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{ee}} = 1.09\,\mathrm{ft}$

16d @ 8" o.c.

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

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

$As := \frac{\left(Z_B\cdot C_o\right)}{vee} = 7.28\,\mathrm{ft}$ $\frac{\left(Z_B\cdot C_o\right)}{E_{ee}} = 9.17\,\mathrm{ft}$

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

WALL FF:

Story Shear due to Wind: $V_{3W} = 9871.01\,\text{lb}$

Story Shear due to Seismic: $F_1 = 9793.22\,\text{lb}$

Bldg Width in direction of Load: $L_t := 33.5 \cdot \text{ft}$

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

Shear Wall Length: $Lff := (7.25 + 7.75)\text{ft} = 15\,\text{ft}$

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: $vff := \dfrac{\dfrac{0.6V_{3W}}{L_t} \cdot \dfrac{L_1}{2}}{Lff}$

Seismic Force: $\rho := 1.0$ $\quad E_{ff} := \dfrac{0.7\rho \cdot \dfrac{0.33F_1}{L_t} \cdot \dfrac{L_1}{2}}{Lff}$

$vff = 126.7\,\text{lb} \cdot \text{ft}^{-1}$ $\quad \dfrac{vff}{C_o} = 126.7\,\text{lb} \cdot \text{ft}^{-1}$

$E_{ff} = 48.4\,\text{lb} \cdot \text{ft}^{-1}$ $\quad \dfrac{E_{ff}}{C_o} = 48.4\,\text{lb} \cdot \text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_{ff} := 7.25 \cdot \text{ft}$ Plate Height: $Pt := 8.5 \cdot \text{ft}$

$W_{ff} := (15 \cdot \text{psf}) \cdot 11.5 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 0\text{ft}$

$DLRff := \dfrac{W_{ff} \cdot L_{ff}}{2}$ $\quad DLRff = 933.44\,\text{lb}$

Chord Force:

$CFff_w := \dfrac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $\quad CFff_w = 1076.97\,\text{lb}$

$CFff_s := \dfrac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}}$ $\quad CFff_s = 411.37\,\text{lb}$

Holdown Force:

$HDFff_w := CFff_w - 0.6 \cdot DLRff = 516.91\,\text{lb}$

$HDFff_s := CFff_s - \left(0.6 - 0.14S_{DS}\right)DLRff = -1.11\,\text{lb}$

No Holdown Required

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

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

$B_n := \dfrac{(Z_N \cdot C_D \cdot C_o)}{vff} = 1.29\,\text{ft}$ $\quad \dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{ff}} = 3.37\,\text{ft}$

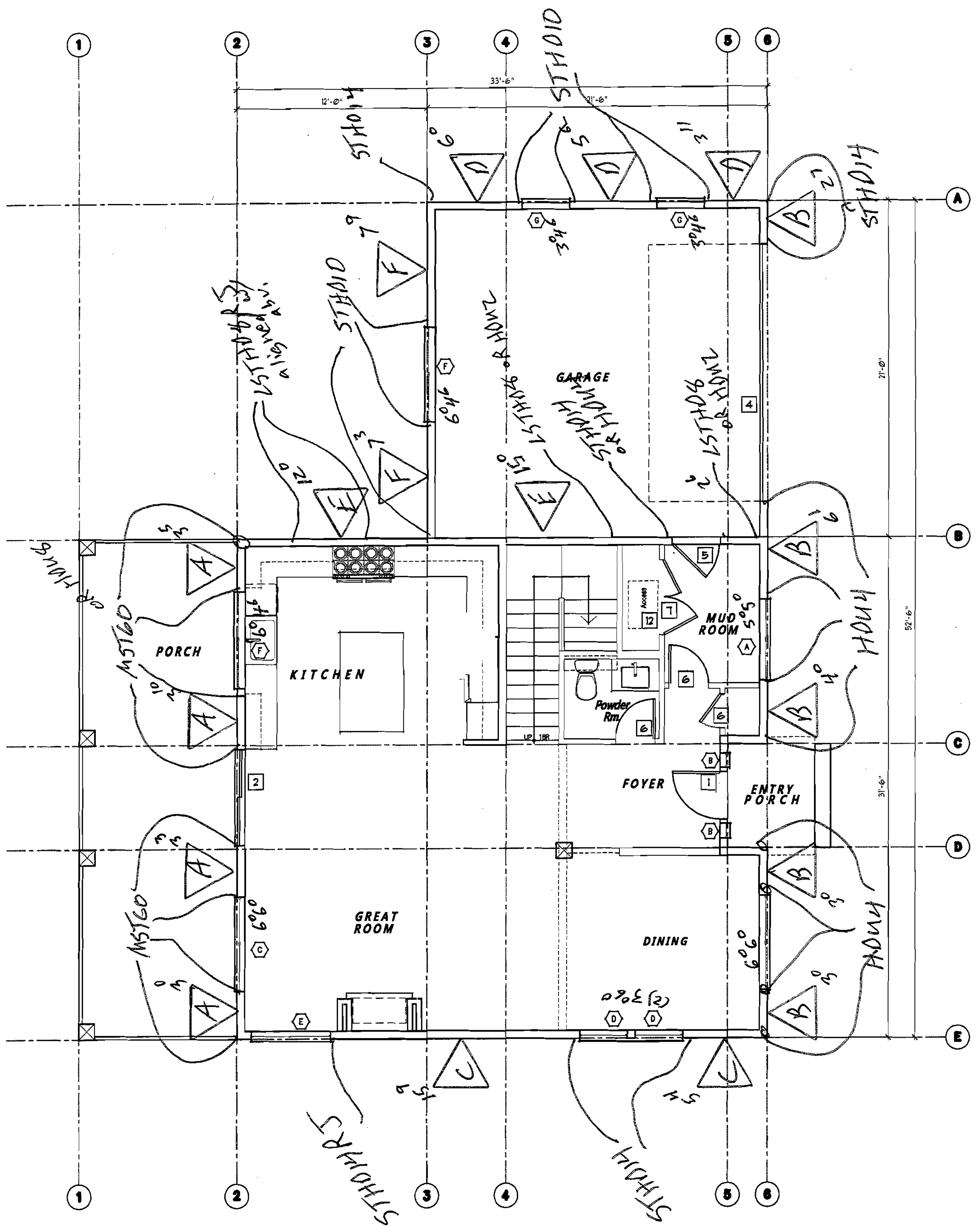
16d @ 12" o.c.

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

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

$A_s := \dfrac{(Z_B \cdot C_o)}{vff} = 10.86\,\text{ft}$ $\quad \dfrac{(Z_B \cdot C_o)}{E_{ff}} = 28.43\,\text{ft}$

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


33'-6"
12'-0"
21'-6"
21'-0"
52'-6"
31'-6"
GARAGE
PORCH
KITCHEN
MUD ROOM
Powder Rm.
FOYER
ENTRY PORCH
GREAT ROOM
DINING
UP 1BR
Access

WALL A:

Story Shear due to Wind: $V_{4W} = 11719.35\,lb$

Story Shear due to Seismic: $F_2 = 8066.6\,lb$

Bldg Width in direction of Load: $L_t := 33.5 \cdot ft$

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

Shear Wall Length: $La := \left[3\left(\frac{6}{10}\right) + 3.25\left(\frac{6.5}{10}\right) + 3.83\left(\frac{7.67}{10}\right) + 3.42\left(\frac{6.83}{10}\right)\right] ft = 9.19\,ft$

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: $va := \dfrac{vaa \cdot Laa + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

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

$va = 459.47\,lb \cdot ft^{-1}$ $\quad \frac{va}{C_o} = 459.47\,lb \cdot ft^{-1}$

$E_a = 360.1\,lb \cdot ft^{-1}$ $\quad \frac{E_a}{C_o} = 360.1\,lb \cdot ft^{-1}$

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

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

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

$DLRa := \frac{W_a \cdot L_a}{2}$ $\quad DLRa = 240\,lb$

Chord Force:

$CFa_w := \frac{va \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $\quad CFa_w = 4594.72\,lb$

$CFa_w + CFaa_w = 6251.18\,lb$

$CFa_s := \frac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a}$ $\quad CFa_s = 3600.98\,lb$

$CFa_s + CFaa_s = 4885.57\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 4450.72\,lb$

$HDFa_w + HDFaa_w = 6003.68\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS}) \cdot DLRa = 3494.92\,lb$

$HDFa_s + HDFaa_s = 4703.29\,lb$

Simpson MST60 Strap

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{va} = 0.36\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.45\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{va} = 2.99\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_a} = 3.82\,ft$

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

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WALL B:

Story Shear due to Wind: $V_{4W} = 11719.35\,lb$ Story Shear due to Seismic: $F_2 = 8066.6\,lb$

Bldg Width in direction of Load: $L_t := 33.5 \cdot ft$ Distance between shear walls: $L_1 := 21.5 \cdot ft$

Shear Wall Length: $Lb := \left[2 \cdot 3\left(\frac{6}{10}\right) + 4\left(\frac{8}{10}\right) + 6.083 + 2\right] ft = 14.88\,ft$

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

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

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

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

$vb = 350.58\,lb \cdot ft^{-1}$ $\dfrac{vb}{C_o} = 350.58\,lb \cdot ft^{-1}$ $E_b = 352.05\,lb \cdot ft^{-1}$ $\dfrac{E_b}{C_o} = 352.05\,lb \cdot ft^{-1}$

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

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

Restraint Panel Height = 10ft Maximum

Restraint Panel Width = 2ft - 0in Minimum

Allowable Shear per Panel = 1125 lbs Seismic & 1575 lbs Wind

Shear per Panel: $V_{s1} := (2ft \cdot E_b) = 704.11\,lb$ O.K.

$V_{s2} := (2ft \cdot vb) = 701.16\,lb$ O.K.

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

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRb := \dfrac{W_b \cdot L_b}{2}$ $DLRb = 165\,lb$

Chord Force:

$CFb_w := \dfrac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_w = 3505.82\,lb$

$CFb_w + CFbb_w = 4444.06\,lb$

$CFb_s := \dfrac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b}$ $CFb_s = 3520.53\,lb$

$CFb_s + CFbb_s = 4606.51\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 3406.82\,lb$

$HDFb_w + HDFbb_w = 4246.06\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14 S_{DS}) \cdot DLRb = 3447.62\,lb$

$HDFb_s + HDFbb_s = 4460.68\,lb$

Simpson HDU4 w/ SB5/8x24 Anchor

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

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

$B_n := \dfrac{(C_D \cdot Z_N \cdot C_o)}{vb} = 0.47\,ft$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.46\,ft$

16d @ 4" o.c.

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

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

$A_s := \dfrac{(Z_B \cdot C_o)}{vb} = 3.92\,ft$ $\dfrac{(Z_B \cdot C_o)}{E_b} = 3.91\,ft$

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

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PROJECT : **3419 72nd Place SE**

WALL C:

Story Shear due to Wind: $V_{2W} = 18384.75\,\text{lb}$ Story Shear due to Seismic: $F_2 = 8066.6\,\text{lb}$

Bldg Width in direction of Load: $L_t := 52.5 \cdot \text{ft}$ Distance between shear walls: $L_1 := 31.5 \cdot \text{ft}$

Shear Wall Length: $Lc := (5.33 + 15.5)\text{ft} = 20.83\,\text{ft}$

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: $vc := \frac{vcc \cdot Lcc + \left(\frac{0.6 V_{2W}}{L_t} \cdot \frac{L_1}{2}\right)}{Lc}$ Seismic Force: $\rho := 1.0$ $E_c := \frac{E_{cc} \cdot Lcc + \left(0.7 \rho \cdot \frac{0.67 F_2}{2}\right)}{Lc}$

$vc = 283.18\,\text{lb} \cdot \text{ft}^{-1}$ $\frac{vc}{C_o} = 283.18\,\text{lb} \cdot \text{ft}^{-1}$ $E_c = 201.06\,\text{lb} \cdot \text{ft}^{-1}$ $\frac{E_c}{C_o} = 201.06\,\text{lb} \cdot \text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_c := 5.33 \cdot \text{ft}$ Plate Height: $Pt := 10 \cdot \text{ft}$

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

$DLRc := \frac{W_c \cdot L_c}{2}$ $DLRc = 426.4\,\text{lb}$

Chord Force:

$CFc_w := \frac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $CFc_w = 2831.81\,\text{lb}$ $CFc_w + CFcc_w = 4074.92\,\text{lb}$

$CFc_s := \frac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $CFc_s = 2010.62\,\text{lb}$ $CFc_s + CFcc_s = 3113.12\,\text{lb}$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 2575.97\,\text{lb}$

$HDFc_w + HDFcc_w = 3515.27\,\text{lb}$

$HDFc_s := CFc_s - \left(0.6 - 0.14 S_{DS}\right) \cdot DLRc = 1822.2\,\text{lb}$

$HDFc_s + HDFcc_s = 2700.95\,\text{lb}$

Simpson STHD14/RJ or HDU4 w/ SB5/8x24 Anchor

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

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

$B_R := \frac{\left(C_D \cdot Z_N \cdot C_o\right)}{vc} = 0.58\,\text{ft}$ $\frac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_c} = 0.81\,\text{ft}$

16d @ 6" o.c.

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

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

$As := \frac{\left(Z_B \cdot C_o\right)}{vc} = 4.86\,\text{ft}$ $\frac{\left(Z_B \cdot C_o\right)}{E_c} = 6.84\,\text{ft}$

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

WALL D:

Story Shear due to Wind: $V_{2W} = 18384.75\,lb$ Story Shear due to Seismic: $F_2 = 8066.6\,lb$

Bldg Width in direction of Load: $L_t := 52.5 \cdot ft$ Distance between shear walls: $L_1 := 21 \cdot ft$

Shear Wall Length: $Ld := \left[6 + 5.5 + 3.92\left(\frac{7.83}{10}\right)\right] ft = 14.57\,ft$

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

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

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

Seismic Force: $\rho := 1.0$ $E_d := \dfrac{E_{dd} \cdot Ldd + \left(0.7\rho \cdot \frac{0.33F_2}{2}\right)}{Ld}$

$vd = 269.91\,lb \cdot ft^{-1}$ $\frac{vd}{C_o} = 269.91\,lb \cdot ft^{-1}$

$E_d = 141.59\,lb \cdot ft^{-1}$ $\frac{E_d}{C_o} = 141.59\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 365 plf
Seismic Capacity = 260 plf

Dead Load Resisting Overturning: $L_d := 3.92 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_d := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 3ft$

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 254.8\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 2699.11\,lb$

$CFd_w + CFdd_w = 4243.67\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 1415.85\,lb$

$CFd_s + CFdd_s = 2427.91\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 2546.23\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 1303.26\,lb$

$HDFd_w + HDFdd_w = 3926.91\,lb$

$HDFd_s + HDFdd_s = 2194.62\,lb$

Simpson STHD14 at corners or STHD10 at Midwall

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 0.6\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 1.15\,ft$

16d @ 6" o.c.

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

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

$A_s := \frac{(Z_B \cdot C_o)}{vd} = 5.1\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 9.72\,ft$

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

WALL E:

Story Shear due to Wind: $V_{2W} = 18384.75\,lb$ Story Shear due to Seismic: $F_2 = 8066.6\,lb$

Bldg Width in direction of Load: $L_t := 52.5 \cdot ft$ Distance between shear walls: $L_1 := 31.5 \cdot ft$ $L_2 := 21ft$

Shear Wall Length: $Le := (12 + 15)ft = 27\,ft$

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

$ve = 364.11\,lb \cdot ft^{-1}$ $\frac{ve}{C_o} = 364.11\,lb \cdot ft^{-1}$ $E_e = 231.52\,lb \cdot ft^{-1}$ $\frac{E_e}{C_o} = 231.52\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_e := 12 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_e := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 6.5ft$ $DLRe := \frac{W_e \cdot L_e}{2}$ $DLRe = 990\,lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_w = 3641.14\,lb$ $CFe_w + CFee_w = 5247.24\,lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e}$ $CFe_s = 2315.16\,lb$ $CFe_s + CFee_s = 3590.77\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 3047.14\,lb$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS}) \cdot DLRe = 1877.69\,lb$

$HDFe_w + HDFee_w = 4344.3\,lb$ $HDFe_s + HDFee_s = 2925.76\,lb$

Simpson STHD14/RJ or HDU4 w/ SB5/8x24 anchor

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{ve} = 0.45\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 0.7\,ft$

16d @ 4" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{ve} = 3.78\,ft$ $\frac{(Z_B \cdot C_o)}{E_e} = 5.94\,ft$

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

WALL F:

Story Shear due to Wind: $V_{4W} = 11719.35\,lb$ Story Shear due to Seismic: $F_2 = 8066.6\,lb$

Bldg Width in direction of Load: $L_t := 33.5 \cdot ft$ Distance between shear walls: $L_1 := 12 \cdot ft$ $L_2 := 21.5ft$

Shear Wall Length: $Lf := (7.25 + 7.75)ft = 15\,ft$

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

$vf = 361.09\,lb \cdot ft^{-1}$ $\frac{vf}{C_o} = 361.09\,lb \cdot ft^{-1}$ $E_f = 236.62\,lb \cdot ft^{-1}$ $\frac{E_f}{C_o} = 236.62\,lb \cdot ft^{-1}$

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

Dead Load Resisting Overturning: $L_f := 7.25 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 398.75\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 3610.89\,lb$ $CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 2366.17\,lb$

$CFf_w + CFff_w = 4687.87\,lb$ $CFf_s + CFff_s = 2777.53\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 3371.64\,lb$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 2189.96\,lb$

$HDFf_w + HDFff_w = 3888.55\,lb$ $HDFf_s + HDFff_s = 2188.85\,lb$

Simpson STHD14 at corners or STHD10 at Midwall

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 0.45\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 0.69\,ft$

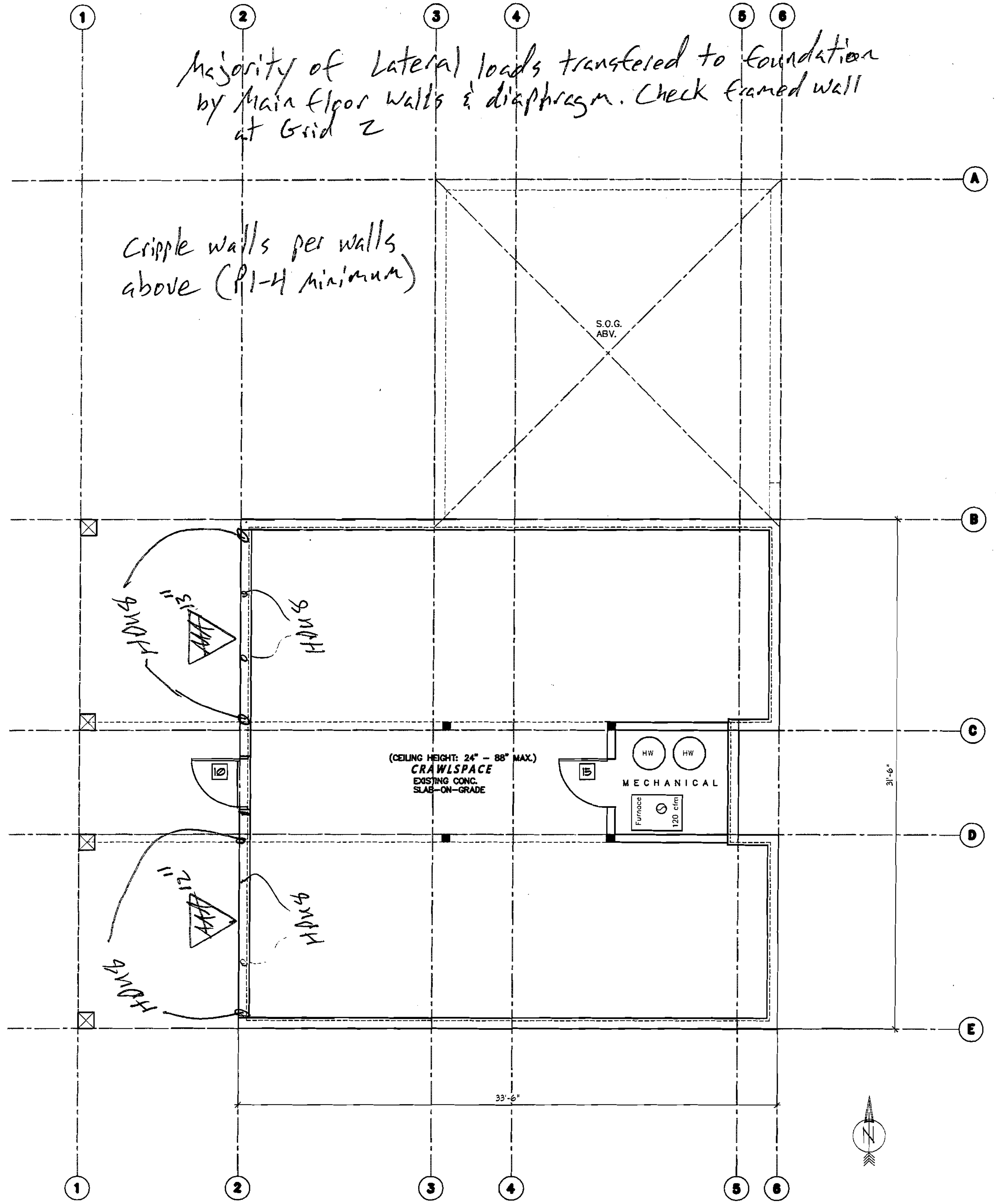
16d @ 4" o.c.

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

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

$A_S := \frac{(Z_B \cdot C_o)}{vf} = 3.81\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 5.82\,ft$

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


Majority of Lateral loads transfered to foundation by Main floor walls & diaphragm. Check framed wall at Grid 2
Cripple walls per walls above (P1-4 minimum)
S.O.G. ABV.
(CEILING HEIGHT: 24" – 88" MAX.)
CRAWLSPACE
EXISTING CONC. SLAB-ON-GRADE
MECHANICAL
HW
HW
Furnace
120 cfm
10
15
HDU8
HDU6
13"
12"
31'-6"
33'-6"
1
2
3
4
5
6
A
B
C
D
E
N

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

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

WALL AAA:

Story Shear due to Wind: $V_{5W} = 3750.19\,\text{lb}$

Story Shear due to Seismic: $F_3 = 1548.81\,\text{lb}$

Bldg Width in direction of Load: $L_t := 12\cdot\text{ft}$

Distance between shear walls: $L_1 := 12\cdot\text{ft}$

Shear Wall Length:

$$Laaa := (12.92 + 13.92)\text{ft} = 26.84\,\text{ft}$$

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: $vaaa := \dfrac{va\cdot La + \left(\dfrac{0.6V_{4W}}{L_t}\cdot\dfrac{L_1}{2}\right)}{Laaa}$

Seismic Force: $\rho := 1.0$ $\quad E_{aaa} := \dfrac{E_a\cdot La + \left[0.7\rho\cdot\left(\dfrac{F_2}{L_t}\cdot\dfrac{L_1}{2}\right)\right]}{Laaa}$

$vaaa = 288.25\,\text{lb}\cdot\text{ft}^{-1}$ $\quad \dfrac{vaaa}{C_o} = 288.25\,\text{lb}\cdot\text{ft}^{-1}$

$E_{aaa} = 228.43\,\text{lb}\cdot\text{ft}^{-1}$ $\quad \dfrac{E_{aaa}}{C_o} = 228.43\,\text{lb}\cdot\text{ft}^{-1}$

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

Dead Load Resisting Overturning: $L_{aaa} := 12.92\cdot\text{ft}$ $\quad$ Plate Height: $Pt := 7\cdot\text{ft}$

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

$DLRaaa := \dfrac{W_{aaa}\cdot L_{aaa}}{2}$ $\quad DLRaaa = 839.8\,\text{lb}$

Chord Force:

$CFaaa_w := \dfrac{vaaa\cdot L_{aaa}\cdot Pt}{C_o\cdot L_{aaa}}$ $\quad CFaaa_w = 2017.72\,\text{lb}$

$CFaaa_w + CFa_w = 6612.44\,\text{lb}$

$CFaaa_s := \dfrac{E_{aaa}\cdot L_{aaa}\cdot Pt}{C_o\cdot L_{aaa}}$ $\quad CFaaa_s = 1599.03\,\text{lb}$

$CFaaa_s + CFa_s = 5200.01\,\text{lb}$

Holdown Force:

$HDFaaa_w := CFaaa_w - 0.6\cdot DLRaaa = 1513.84\,\text{lb}$

$HDFaaa_w + HDFa_w = 5964.56\,\text{lb}$

$HDFaaa_s := CFaaa_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRaaa = 1227.93\,\text{lb}$

$HDFaaa_s + HDFa_s = 4722.85\,\text{lb}$

Simpson HDU8 w/ SB7/8x24 anchor

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

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

$B_B := \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{vaaa} = 0.57\,\text{ft}$ $\quad \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{aaa}} = 0.71\,\text{ft}$

16d @ 6" o.c.

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

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

$As := \dfrac{\left(Z_B\cdot C_o\right)}{vaaa} = 4.77\,\text{ft}$ $\quad \dfrac{\left(Z_B\cdot C_o\right)}{E_{aaa}} = 6.02\,\text{ft}$

5/8" A.B. @ 54" 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:

$vaa \cdot \frac{Laa}{31ft} = 95.53\,lb \cdot ft^{-1}$ $\quad E_{aa} \cdot \frac{Laa}{31ft} = 74.08\,lb \cdot ft^{-1}$

Wall Lines DD:

$vdd \cdot \frac{Ldd}{21ft} = 82.2\,lb \cdot ft^{-1}$ $\quad E_{dd} \cdot \frac{Ldd}{21ft} = 53.86\,lb \cdot ft^{-1}$

Wall Lines BB:

$vbb \cdot \frac{Lbb}{52ft} = 56.95\,lb \cdot ft^{-1}$ $\quad E_{bb} \cdot \frac{Lbb}{52ft} = 65.92\,lb \cdot ft^{-1}$

Wall Lines EE:

$vee \cdot \frac{Lee}{33ft} = 130.78\,lb \cdot ft^{-1}$ $\quad E_{ee} \cdot \frac{Lee}{33ft} = 103.87\,lb \cdot ft^{-1}$

Wall Lines CC:

$vcc \cdot \frac{Lcc}{33ft} = 78.47\,lb \cdot ft^{-1}$ $\quad E_{cc} \cdot \frac{Lcc}{33ft} = 69.59\,lb \cdot ft^{-1}$

Wall Lines A:

$\frac{va \cdot La - vaa \cdot Laa}{31ft} = 40.63\,lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La - E_{aa} \cdot Laa}{31ft} = 32.62\,lb \cdot ft^{-1}$ $\quad \frac{va \cdot La}{31ft} = 136.15\,lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La}{31ft} = 106.7\,lb \cdot ft^{-1}$

Wall Lines B:

$\frac{vb \cdot Lb - vbb \cdot Lbb}{52ft} = 43.39\,lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{52ft} = 34.85\,lb \cdot ft^{-1}$ $\quad \frac{vb \cdot Lb}{52ft} = 100.34\,lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb}{52ft} = 100.76\,lb \cdot ft^{-1}$

Wall Lines C:

$\frac{vc \cdot Lc - vcc \cdot Lcc}{33ft} = 100.28\,lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc - E_{cc} \cdot Lcc}{33ft} = 57.32\,lb \cdot ft^{-1}$ $\quad \frac{vc \cdot Lc}{33ft} = 178.75\,lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc}{33ft} = 126.91\,lb \cdot ft^{-1}$

Wall Lines D:

$\frac{vd \cdot Ld - vdd \cdot Ldd}{21ft} = 105.06\,lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{21ft} = 44.37\,lb \cdot ft^{-1}$ $\quad \frac{vd \cdot Ld}{21ft} = 187.26\,lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld}{21ft} = 98.23\,lb \cdot ft^{-1}$

Wall Line E:

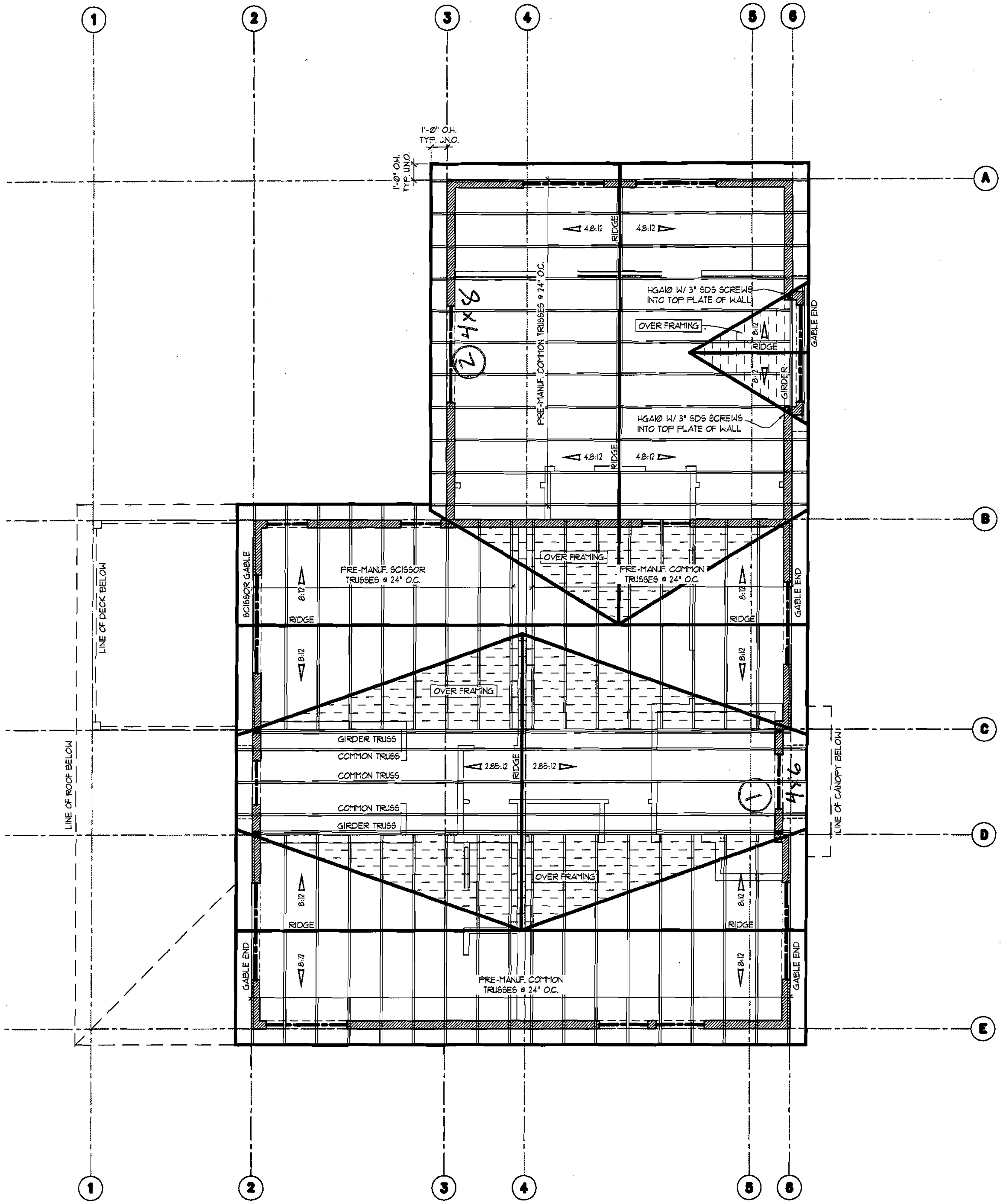
$\frac{ve \cdot Le - vee \cdot Lee}{33ft} = 167.13\,lb \cdot ft^{-1}$ $\quad \frac{E_e \cdot Le - E_{ee} \cdot Lee}{33ft} = 85.55\,lb \cdot ft^{-1}$ $\quad \frac{ve \cdot Le}{33ft} = 297.91\,lb \cdot ft^{-1}$ $\quad \frac{E_e \cdot Le}{33ft} = 189.42\,lb \cdot ft^{-1}$

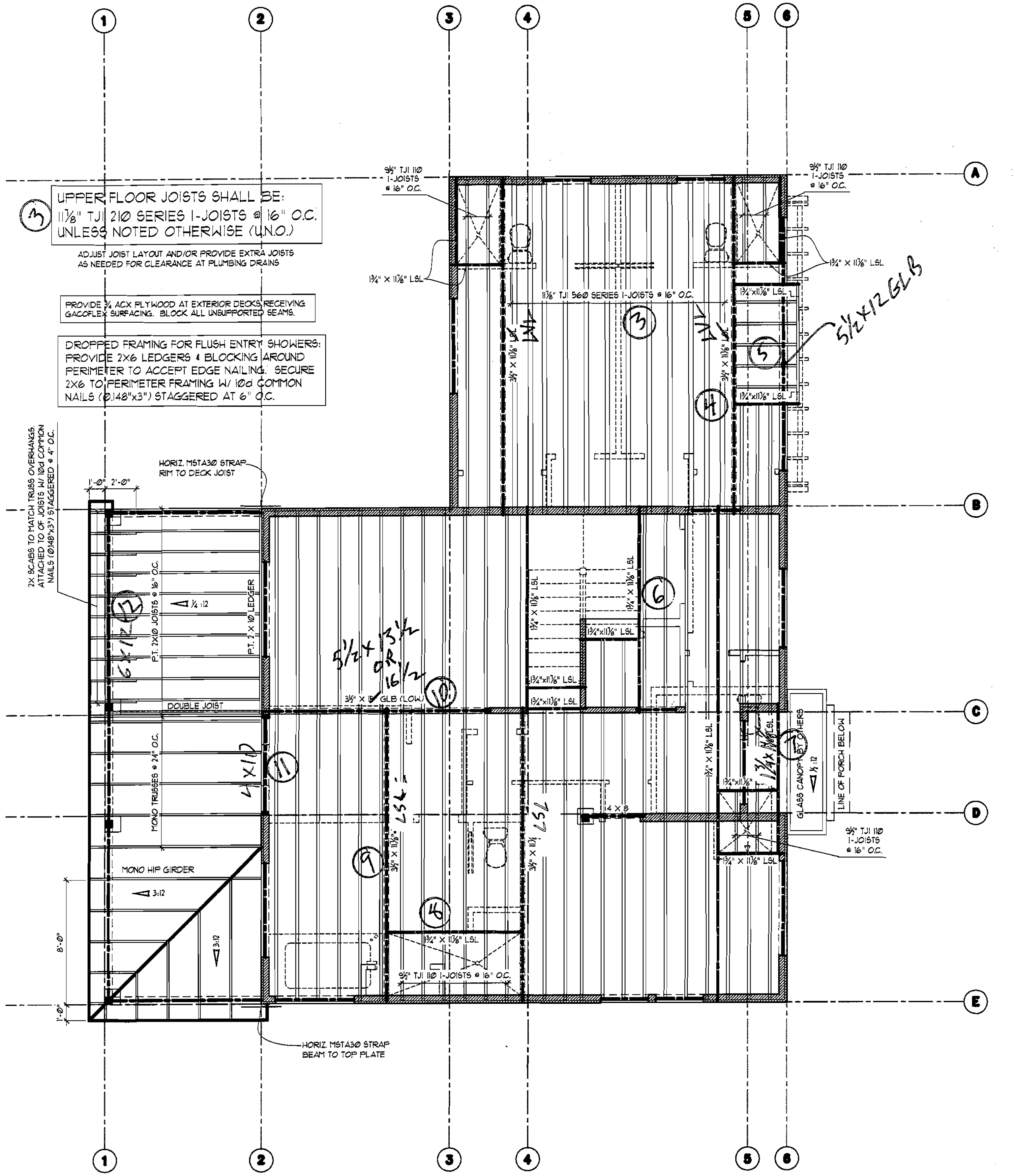
Wall Line F:

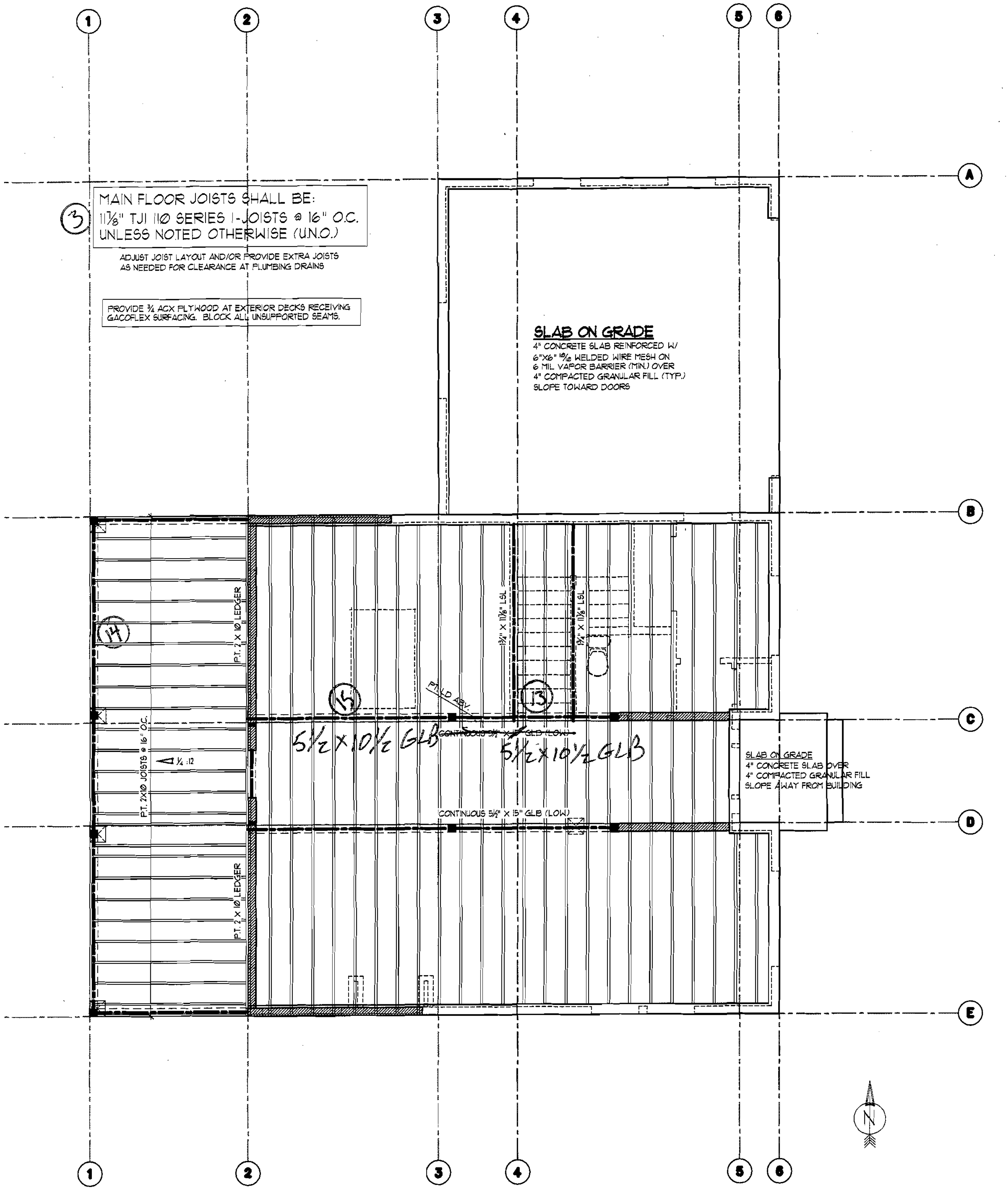
$\frac{vf \cdot Lf - vff \cdot Lff}{34ft} = 103.41\,lb \cdot ft^{-1}$ $\quad \frac{E_f \cdot Lf - E_{ff} \cdot Lff}{34ft} = 83.04\,lb \cdot ft^{-1}$ $\quad \frac{vf \cdot Lf}{34ft} = 159.3\,lb \cdot ft^{-1}$ $\quad \frac{E_f \cdot Lf}{34ft} = 104.39\,lb \cdot ft^{-1}$

Wall Lines AAA:

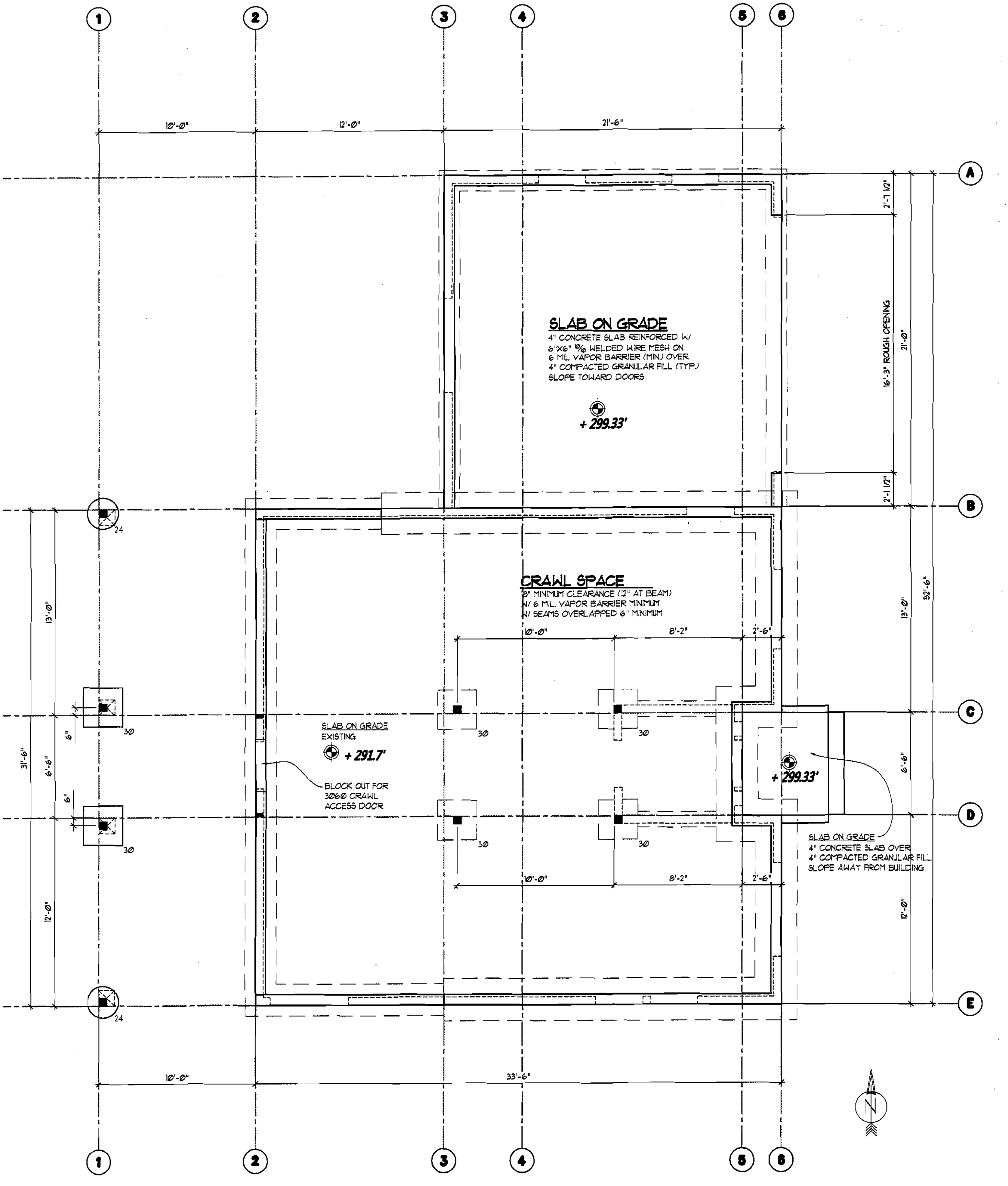
$vaaa \cdot \frac{Laaa}{31ft} = 249.56\,lb \cdot ft^{-1}$ $\quad E_{aaa} \cdot \frac{Laaa}{31ft} = 197.78\,lb \cdot ft^{-1}$


1
2
3
4
5
6
A
B
C
D
E
1'-0" O.H. TYP. U.N.O.
1'-0" O.H. TYP. U.N.O.
4.8:12
RIDGE
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
HGA10 W/ 3" SDS SCREWS INTO TOP PLATE OF WALL
OVER FRAMING
8:12
RIDGE
GIRDER
GABLE END
4x6
2
SCISSOR GABLE
PRE-MANUF. SCISSOR TRUSSES @ 24" O.C.
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
LINE OF DECK BELOW
LINE OF ROOF BELOW
GIRDER TRUSS
COMMON TRUSS
2.85:12
LINE OF CANOPY BELOW
1


UPPER FLOOR JOISTS SHALL BE:
11⅞" TJI 210 SERIES I-JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
ADJUST JOIST LAYOUT AND/OR PROVIDE EXTRA JOISTS AS NEEDED FOR CLEARANCE AT PLUMBING DRAINS
PROVIDE ¾ ACX PLYWOOD AT EXTERIOR DECKS RECEIVING GACOFLEX SURFACING. BLOCK ALL UNSUPPORTED SEAMS.
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.
2X SCABS TO MATCH TRUSS OVERHANGS, ATTACHED TO OF JOISTS W/ 10d COMMON NAILS (0.148"x3") STAGGERED @ 4" O.C.
HORIZ. MSTA30 STRAP RIM TO DECK JOIST
HORIZ. MSTA30 STRAP BEAM TO TOP PLATE
9½" TJI 110 I-JOISTS @ 16" O.C.
1¾" × 11⅞" LSL
11⅞" TJI 560 SERIES I-JOISTS @ 16" O.C.
3½" × 11⅞" LSL
1¾"x11⅞" LSL
5½x12 GLB
P.T. 2X10 JOISTS @ 16" O.C.
P.T. 2 X 10 LEDGER
¼ :12
6x12
DOUBLE JOIST
5½ x 13½ OR 16½
3½" X 15 GLB (LOW)
MONO TRUSSES @ 24" O.C.
4x10
LSL
4 X 8
GLASS CANOPY BY OTHERS
½ :12
LINE OF PORCH BELOW
MONO HIP GIRDER
3:12
8'-0"
1'-0"
2'-0"
9½" TJI 110 I-JOISTS @ 16" O.C.
3½" × 11¼
1¾"x11⅞"
3¼x
1 2 3 4 5 6
A B C D E



MAIN FLOOR JOISTS SHALL BE:
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
3
ADJUST JOIST LAYOUT AND/OR PROVIDE EXTRA JOISTS AS NEEDED FOR CLEARANCE AT PLUMBING DRAINS
PROVIDE ¾ ACX PLYWOOD AT EXTERIOR DECKS RECEIVING GACOFLEX SURFACING. BLOCK ALL UNSUPPORTED SEAMS.
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
14
P.T. 2 X 10 LEDGER
P.T. 2X10 JOISTS @ 16" O.C.
¼ :12
P.T. 2 X 10 LEDGER
1¾" X 11⅞" LSL
1¾" X 11⅞" LSL
PLLD ABV.
15
13
5½ X 10½ GLB
CONTINUOUS 5½" X 15" GLB (LOW)
5½ X 10½ GLB
CONTINUOUS 5½" X 15" GLB (LOW)
SLAB ON GRADE
4" CONCRETE SLAB OVER 4" COMPACTED GRANULAR FILL SLOPE AWAY FROM BUILDING
N
1 2 3 4 5 6
A B C D E



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
+ 299.33'
CRAWL SPACE
18" MINIMUM CLEARANCE (12" AT BEAM)
W/ 6 MIL. VAPOR BARRIER MINIMUM
W/ SEAMS OVERLAPPED 6" MINIMUM
SLAB ON GRADE
EXISTING
+ 291.7'
BLOCK OUT FOR
3060 CRAWL
ACCESS DOOR
+ 299.33'
SLAB ON GRADE
4" CONCRETE SLAB OVER
4" COMPACTED GRANULAR FILL
SLOPE AWAY FROM BUILDING
16'-3" ROUGH OPENING
10'-0"
12'-0"
21'-6"
33'-6"
52'-6"
31'-6"

MYERS ENGINEERING

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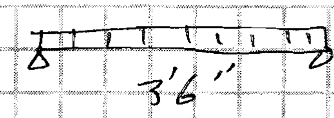
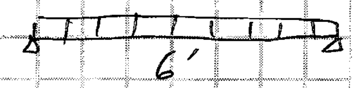
myengineer@centurytel.net

① $w_D = 15psf(36'/2) = 270plf$
$w_S = 25psf(36'/2) = 450plf$



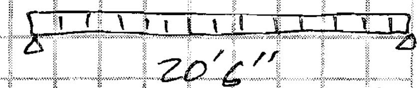
4x6

② $w_D = 15psf(24'/2) = 180plf$
$w_S = 25psf(24'/2) = 300plf$



4x8

③ $w_D = 15psf$
$w_L = 40psf$



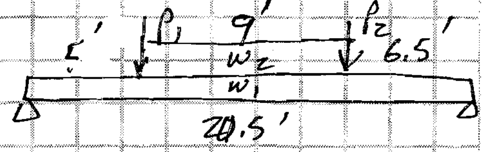
11 7/8 TJI 560 @ 16" o.c.
(210 series @ 19'3" span)

④ $w_{D_1} = 15plf$
$w_{L_1} = 40plf$

$P_1 = 70^{\#} DL + 180^{\#} LL$

$P_2 = 80^{\#} DL + 210^{\#} LL$

$w_{D2} = 15psf(3'/2) = 23plf$
$w_{L2} = 40psf(3'/2) = 60plf$

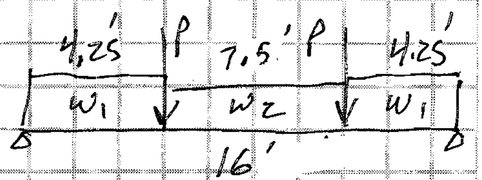


3 1/2 x 11 7/8 LVL

⑤ $w_{D_1} = 15psf(24'/2 + 1') + 12psf(8.5') = 300plf$
$w_{L_1} = 40plf$
$w_{S_1} = 25psf(24'/2) = 300plf$

$P = 760^{\#} DL + 210^{\#} LL + 1130^{\#} SL$

$w_{D2} = 15psf(5'/2 + 1') + 12psf(8.5') = 155plf$
$w_{L2} = 40psf(5'/2) = 100plf$
$w_{S2} = 25plf$



5 1/2 x 12 GLB

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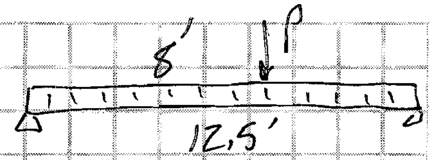
(6) $w_{D1} = 15plf$
$w_{L1} = 40plf$

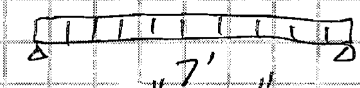
$P = 135^{\#} DL + 360^{\#} LL$



$1\frac{3}{4} \times 11\frac{7}{8}$ LSL

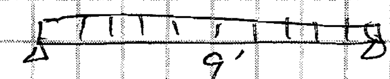
(7) $w_{D1} = 270plf + 12psf(8.5') = 372plf$
$w_{S1} = 450plf$

$w_{D2} = 15psf(5'/2) = 37.5plf$
$w_{L2} = 40psf(2'/2) = 40plf$
$w_{S2} = 25psf(3'/2) = 37.5plf$



$1\frac{3}{4} \times 11\frac{7}{8}$ LSL

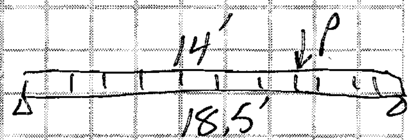
(8) $w_D = 15psf(19'/2) = 142.5plf$
$w_L = 40psf(19'/2) = 380plf$



$1\frac{3}{4} \times 11\frac{7}{8}$ LSL

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

$P = 645^{\#} DL + 1710^{\#} LL$ from (8)

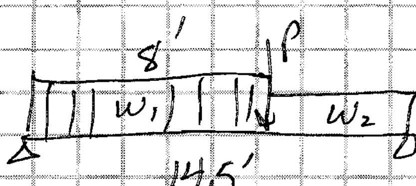


$3\frac{1}{2} \times 11\frac{7}{8}$ LVL

(10) $w_{D1} = 15psf(31'/2) = 232.5plf$
$w_{L1} = 40psf(31'/2) = 620plf$

$w_{D2} = 15psf(27'/2) = 202.5plf$
$w_{L2} = 40psf(27'/2) = 540plf$

$P = 300^{\#} DL + 790^{\#} LL$ from (9)



$3\frac{1}{2} \times 16\frac{1}{2}$ GLB
OR
$5\frac{1}{2} \times 13\frac{1}{2}$ GLB

FOR RFA/RKK
JOB 3415 72nd

DATE 6-7-22
BY MM

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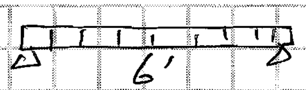
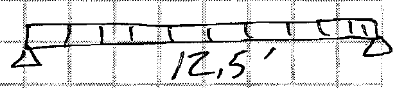
(11) $w_D = 270plf + 15psf(1' + 10'/2) + 12psf(8.5') = 462plf$
$w_L = 40psf(1') = 40plf$
$w_S = 450plf + 25psf(10'/2) = 575plf$



4x10
minimum

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



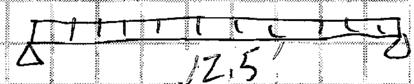
6x12

(13) $w_D = 15psf(20'/2 + 20'/2) = 300plf$
$w_L = 40psf(20'/2 + 20'/2) = 800plf$

$P = 1780^{\#}DL + 4730^{\#}LL$ from (10)

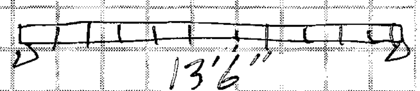
5½x10½GLB

(14) $w_D = 10psf(10'/2) = 50plf$
$w_L = 60psf(10'/2) = 300plf$



6x12

(15) $w_D = 15psf(20'/2) = 150plf$
$w_L = 40psf(20'/2) = 400plf$



5½x10½GLB

FOR RFA/RKK
JOB 3419 72nd

DATE 6-7-22
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

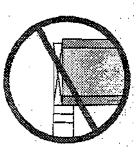
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

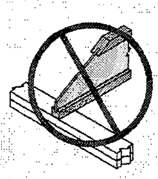
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor 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.

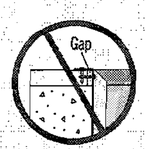
These Conditions Are NOT Permitted:



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: 3419 72nd PL SE.ec6

LIC# : KW-06015659, Build:20.22.5.16 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

Description :

Wood Beam Design : 1. Short Header long span

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

BEAM Size : **4x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

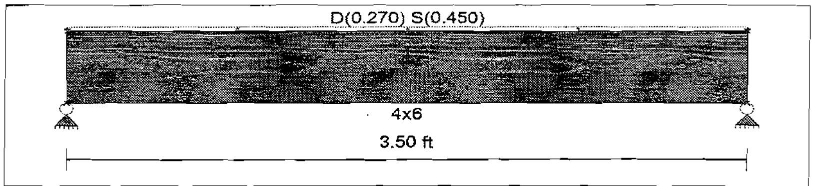
Wood Species : DouglasFir-Larch Wood Grade : No.2

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 = 0.270, S = 0.450 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.559** : 1
fb : Actual : 749.75 psi at 1.750 ft in Span # 1
Fb : Allowable : 1,340.20 psi
Load Comb : +D+S

Max fv/FvRatio = **0.351** : 1
fv : Actual : 72.65 psi at 3.045 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.47			0.79			
Right Support	0.47			0.79			

Max Deflections

Transient Downward	0.020 in	Total Downward	0.031 in
Ratio	2134	Ratio	1334
	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 6ft Worst Case

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

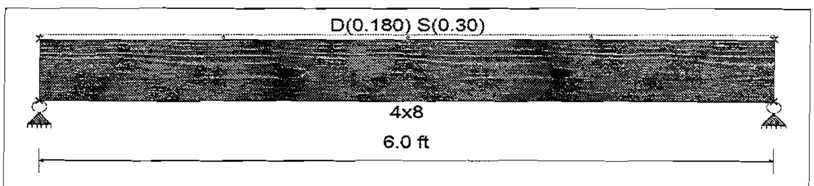
Wood Species : DouglasFir-Larch Wood Grade : No.2

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 = 0.180, S = 0.30 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.634** : 1
fb : Actual : 845.36 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,333.02 psi
Load Comb : +D+S

Max fv/FvRatio = **0.329** : 1
fv : Actual : 68.10 psi at 5.400 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.54			0.90			
Right Support	0.54			0.90			

Max Deflections

Transient Downward	0.049 in	Total Downward	0.079 in
Ratio	1455	Ratio	909
	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. Floor beam at showers over Garage

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

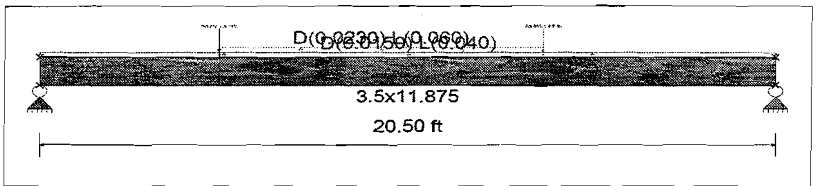
BEAM Size : **3.5x11.875, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist			Wood Grade :	MicroLam LVL 1.9 E			
Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density 42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi	

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
Unif Load: D = 0.0230, L = 0.060 k/ft, 5.0 to 14.0 ft, Trib= 1.0 ft
1Point: D = 0.070, L = 0.180 k @ 5.0 ft
2Point: D = 0.080, L = 0.210 k @ 14.0 ft

Design Summary



Max fb/Fb Ratio = **0.416** : 1
fb : Actual : 1,082.63 psi at 10.250 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L
Max fv/FvRatio = **0.151** : 1
fv : Actual : 43.05 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.34		0.90				
Right Support	0.32		0.85				

Max Deflections

Transient Downward	0.436 in	Total Downward	0.602 in
Ratio	564	Ratio	408
	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 : 5. Garage Door Header

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

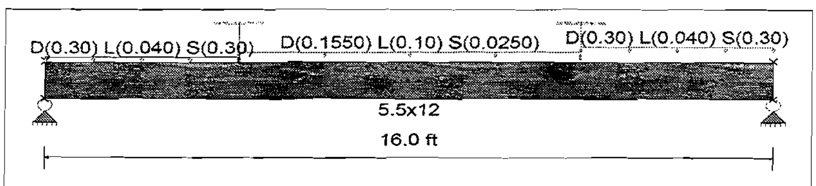
BEAM Size : **5.5x12, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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 = 0.30, L = 0.040, S = 0.30 k/ft, 0.0 ft to 4.250 ft, Trib= 1.0 ft
Unif Load: D = 0.1550, L = 0.10, S = 0.0250 k/ft, 4.250 to 11.750 ft, Trib= 1.0 ft
Unif Load: D = 0.30, L = 0.040, S = 0.30 k/ft, 11.750 to 16.0 ft, Trib= 1.0 ft
1Point: D = 0.760, L = 0.210, S = 1.130 k @ 4.250 ft
2Point: D = 0.760, L = 0.210, S = 1.130 k @ 11.750 ft

Design Summary



Max fb/Fb Ratio = **0.615** : 1
fb : Actual : 1,657.01 psi at 8.000 ft in Span # 1
Fb : Allowable : 2,693.34 psi
Load Comb : +D+0.750L+0.750S
Max fv/FvRatio = **0.339** : 1
fv : Actual : 103.16 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.62		0.76	2.50			
Right Support	2.62		0.76	2.50			

Max Deflections

Transient Downward	0.288 in	Total Downward	0.626 in
Ratio	666	Ratio	306
	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 : 6. Rim Beam at Stair

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

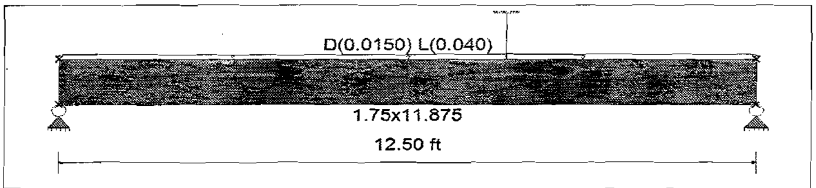
BEAM Size : **1.75x11.875, TimberStrand LSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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 = 0.1350, L = 0.360 k @ 8.0 ft

Design Summary



Max fb/Fb Ratio = **0.833** : 1
fb : Actual : 704.78 psi at 8.000 ft in Span # 1
Fb : Allowable : 846.15 psi
Load Comb : +D+L
Max fv/FvRatio = **0.142** : 1
fv : Actual : 43.87 psi at 11.542 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.14		0.38				
Right Support	0.18		0.48				

Max Deflections

Transient Downward	0.119 in	Total Downward	0.163 in
Ratio	1264	Ratio	919
	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 : 7. Rim Beam over Entry Porch

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

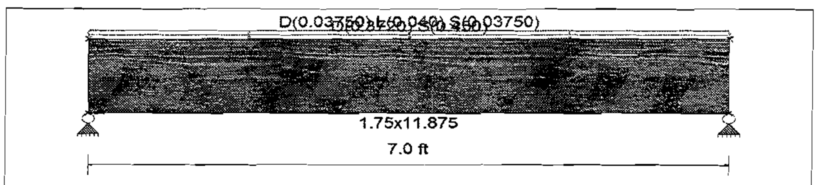
BEAM Size : **1.75x11.875, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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 = 0.3720, S = 0.450 k/ft, Trib= 1.0 ft
Unif Load: D = 0.03750, L = 0.040, S = 0.03750 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.600** : 1
fb : Actual : 1,602.97 psi at 3.500 ft in Span # 1
Fb : Allowable : 2,673.75 psi
Load Comb : +D+S
Max fv/FvRatio = **0.458** : 1
fv : Actual : 163.16 psi at 6.020 ft in Span # 1
Fv : Allowable : 356.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.43		0.14	1.71			
Right Support	1.43		0.14	1.71			

Max Deflections

Transient Downward	0.070 in	Total Downward	0.129 in
Ratio	1200	Ratio	652
	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: 3419 72nd PL SE.ec6

LIC# : KW-06015659, Build:20.22.5.16 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

Wood Beam Design : 8. Floor beam at Shower

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

BEAM Size : **1.75x11.875, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

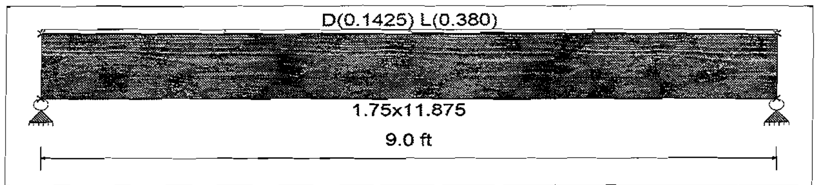
Wood Species : iLevel Truss Joist   Wood Grade : TimberStrand LSL 1.55E

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 = 0.1425, L = 0.380 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.664** : 1
fb : Actual : 1,543.51 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,325.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.431** : 1
fv : Actual : 133.51 psi at 0.000 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.64		1.71				
Right Support	0.64		1.71				

Max Deflections

Transient Downward	0.149 in	Total Downward	0.205 in
Ratio	724	Ratio	527
	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 : 9. Floor beam supporting beam 8

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

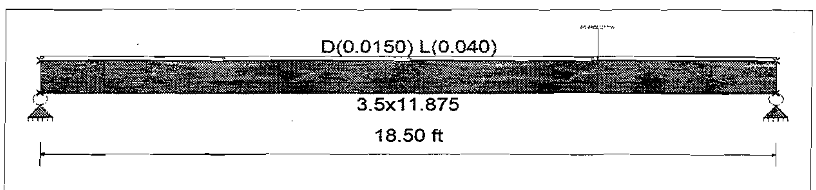
Wood Species : iLevel Truss Joist   Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi		

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
1Point: D = 0.6450, L = 1.710 k @ 14.0 ft

Design Summary



Max fb/Fb Ratio = **0.547** : 1
fb : Actual : 1,422.58 psi at 13.998 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.283** : 1
fv : Actual : 80.72 psi at 17.513 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.30		0.79				
Right Support	0.63		1.66				

Max Deflections

Transient Downward	0.401 in	Total Downward	0.552 in
Ratio	553	Ratio	401
	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: 3419 72nd PL SE.ec6

LIC# : KW-06015659, Build:20.22.5.16 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

Wood Beam Design : 10. Beam over Kitchen

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

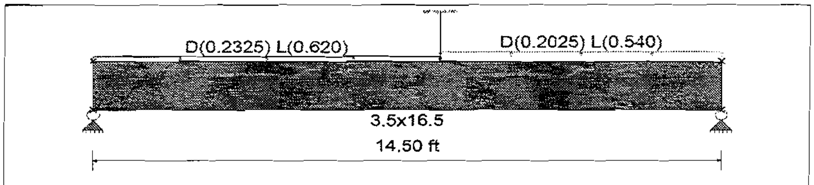
BEAM Size : **3.5x16.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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 = 0.2325, L = 0.620 k/ft, 0.0 ft to 8.0 ft, Trib= 1.0 ft
Unif Load: D = 0.2025, L = 0.540 k/ft, 8.0 to 14.50 ft, Trib= 1.0 ft
1Point: D = 0.30, L = 0.790 k @ 8.0 ft



Design Summary

Max fb/Fb Ratio = **0.932** : 1
fb : Actual : 1,877.58 psi at 7.637 ft in Span # 1
Fb : Allowable : 2,014.45 psi
Load Comb : +D+L
Max fv/FvRatio = **0.525** : 1
fv : Actual : 139.10 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.78		4.73				
Right Support	1.70		4.53				

Max Deflections

Transient Downward	0.285 in	Total Downward	0.392 in
Ratio	610	Ratio	443
	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 : 10. Beam over Kitchen

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

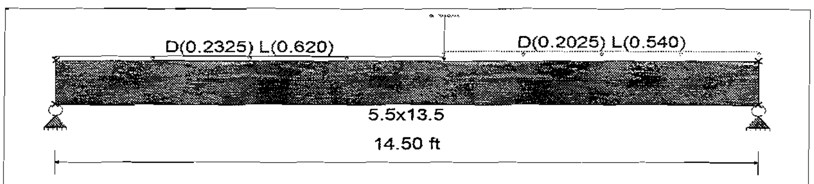
BEAM Size : **5.5x13.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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 = 0.2325, L = 0.620 k/ft, 0.0 ft to 8.0 ft, Trib= 1.0 ft
Unif Load: D = 0.2025, L = 0.540 k/ft, 8.0 to 14.50 ft, Trib= 1.0 ft
1Point: D = 0.30, L = 0.790 k @ 8.0 ft



Design Summary

Max fb/Fb Ratio = **0.759** : 1
fb : Actual : 1,784.86 psi at 7.637 ft in Span # 1
Fb : Allowable : 2,350.07 psi
Load Comb : +D+L
Max fv/FvRatio = **0.424** : 1
fv : Actual : 112.35 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.78		4.73				
Right Support	1.70		4.53				

Max Deflections

Transient Downward	0.331 in	Total Downward	0.456 in
Ratio	525	Ratio	381
	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 : 11. Header at SGD

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

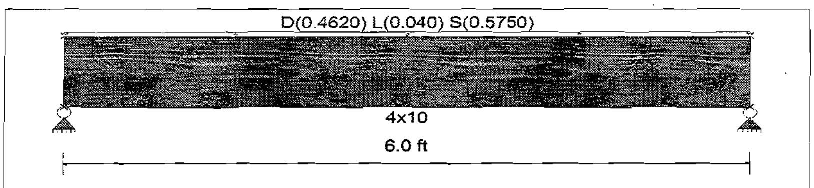
BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

Wood Species : DouglasFir-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 = 0.4620, L = 0.040, S = 0.5750 k/ft, Trib= 1.0 ft



Design Summary

Max fb/Fb Ratio = **0.914** : 1
fb : Actual : 1,121.95 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,227.30 psi
Load Comb : +D+S

Max fv/FvRatio = **0.520** : 1
fv : Actual : 107.62 psi at 5.240 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.39		0.12	1.73			
Right Support	1.39		0.12	1.73			

Max Deflections

Transient Downward	0.046 in	Total Downward	0.082 in
Ratio	1577	Ratio	874
	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. Upper Deck/Roof Beam

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

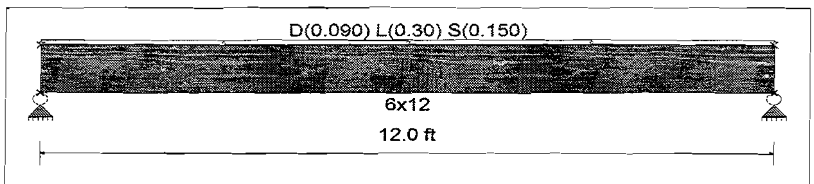
BEAM Size : **6x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

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

Fb - Tension	875 psi	Fc - Prll	600 psi	Fv	170 psi	Ebend- xx	1300 ksi	Density	31.21 pcf
Fb - Compr	875 psi	Fc - Perp	625 psi	Ft	425 psi	Eminbend - xx	470 ksi		

Applied Loads

Unif Load: D = 0.090, L = 0.30, S = 0.150 k/ft, Trib= 1.0 ft



Design Summary

Max fb/Fb Ratio = **0.802** : 1
fb : Actual : 694.88 psi at 6.000 ft in Span # 1
Fb : Allowable : 866.91 psi
Load Comb : +D+L

Max fv/FvRatio = **0.276** : 1
fv : Actual : 46.98 psi at 11.080 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.54		1.80	0.90			
Right Support	0.54		1.80	0.90			

Max Deflections

Transient Downward	0.155 in	Total Downward	0.221 in
Ratio	927	Ratio	650
	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:

Description :

Wood Beam Design : 13. Beam in Crawl East of Grid 3

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

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

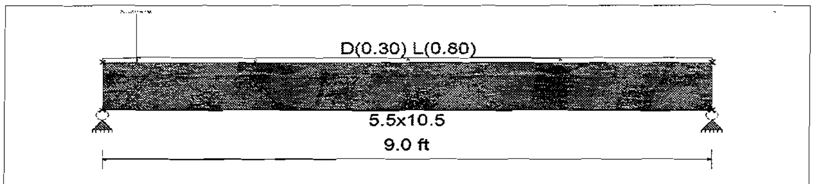
Wood Species : DF/DF Wood Grade : 24F-V4

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 = 0.30, L = 0.80 k/ft, Trib= 1.0 ft
1Point: D = 1.780, L = 4.730 k @ 0.50 ft

Design Summary



Max fb/Fb Ratio = **0.640** : 1
fb : Actual : 1,522.76 psi at 4.170 ft in Span # 1
Fb : Allowable : 2,378.95 psi
Load Comb : +D+L

Max fv/FvRatio = **0.427** : 1
fv : Actual : 113.11 psi at 8.130 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.03		8.07				
Right Support	1.45		3.86				

Max Deflections

Transient Downward	0.146 in	Total Downward	0.201 in
Ratio	738	Ratio	537
	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 : 14. Main Floor Deck Beam

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

BEAM Size : **6x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2018 Load Combinations, Major Axis Bending

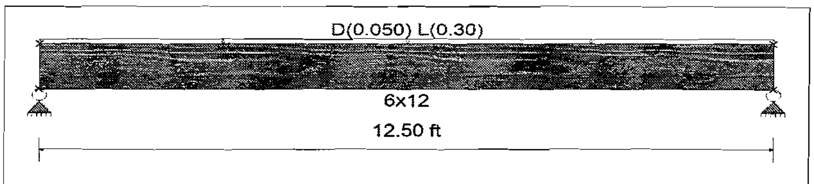
Wood Species : DouglasFir-Larch Wood Grade : No.2

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 = 0.050, L = 0.30 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.758** : 1
fb : Actual : 676.66 psi at 6.250 ft in Span # 1
Fb : Allowable : 892.97 psi
Load Comb : +D+L

Max fv/FvRatio = **0.246** : 1
fv : Actual : 44.27 psi at 11.583 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.31		1.88				
Right Support	0.31		1.88				

Max Deflections

Transient Downward	0.149 in	Total Downward	0.173 in
Ratio	1009	Ratio	865
	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: 3419 72nd PL SE.ec6

LIC# : KW-06015659, Build:20.22.5.16 MYERS ENGINEERING (c) ENERCALC INC 1983-2022

Wood Beam Design : 15. Beam in Crawl West of Grid 3

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

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

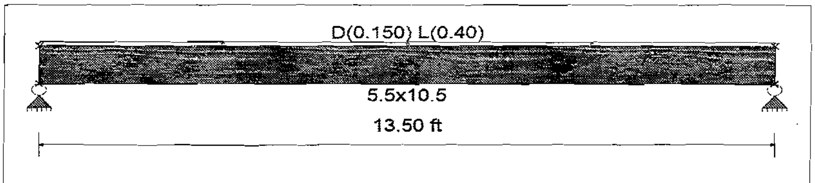
Wood Species : DF/DF Wood Grade : 24F-V4

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 = 0.150, L = 0.40 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.628** : 1
fb : Actual : 1,487.76 psi at 6.750 ft in Span # 1
Fb : Allowable : 2,367.60 psi
Load Comb : +D+L

Max fv/FvRatio = **0.318** : 1
fv : Actual : 84.21 psi at 12.645 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.01		2.70				
Right Support	1.01		2.70				

Max Deflections

Transient Downward	0.315 in	Total Downward	0.433 in
Ratio	514	Ratio	374
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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Maximum Load For 4x4 HF#2 Treated Post

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

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

$E' := 1235000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

4x4 Treated Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

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

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

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

$C_p = 0.6$

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

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Project Name/Number : cantilever wa
Title **8ft Stem**
Dsgnr: **Mark Myers, PE**
Description....

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Date: 28 MAR 2016

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Cantilevered Retaining Wall

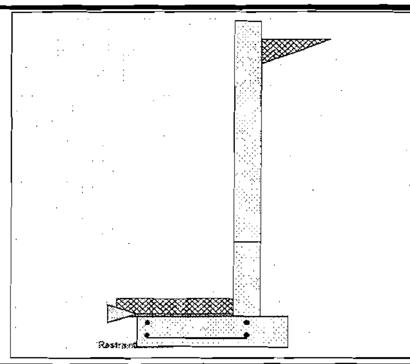
Code: IBC 2018,ACI 318-14,TMS 402-16

Criteria

Retained Height	=	7.50 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	125.00 pcf
Soil Density, Toe	=	125.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	1.55 OK
Slab Resists All Sliding !		
Total Bearing Load	=	2,029 lbs
...resultant ecc.	=	11.06 in
Soil Pressure @ Toe	=	1,484 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	2,077 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	25.6 psi OK
Footing Shear @ Heel	=	10.3 psi OK
Allowable	=	75.0 psi

Sliding Calcs

Lateral Sliding Force	=	1,215.3 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2018,ACI
Dead Load	1.400
Live Load	1.700
Earth, H	1.700
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	2.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 4	# 4
Rebar Spacing	=	12.00	6.00
Rebar Placed at	=	Center	Center
Design Data			
fb/FB + fa/Fa	=	**0.487**	**0.658**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	899.9	1,673.4
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	1,649.9	4,183.6
Moment.....Allowable	ft-# =	3,387.6	6,350.4
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	18.7	34.9
Shear.....Allowable	psi =	75.0	75.0
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	4.00	4.00
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0996 in2/ft		
(4/3) * As :	0.1328 in2/ft	Min Stem T&S Reinf Area 1.152 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.2525 in2/ft		
(4/3) * As :	0.3367 in2/ft	Min Stem T&S Reinf Area 0.384 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.2525 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.4 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	2.33 ft
Heel Width	=	1.33
Total Footing Width	=	3.67
Footing Thickness	=	10.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.92 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	2,077	0 psf
Mu' : Upward	=	48,545	0 ft-#
Mu' : Downward	=	8,573	330 ft-#
Mu: Design	=	3,331	330 ft-#
Actual 1-Way Shear	=	25.62	10.32 psi
Allow 1-Way Shear	=	75.00	40.00 psi
Toe Reinforcing	=	# 4 @ 9.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings
Toe: #4@ 11.11 in, #5@ 17.22 in, #6@ 24.44 in, #7@ 33.33 in, #8@ 43.88 in, #9@ 5
Heel: phiMn = phi'5'lambda'sqrt(fc)'Sm
Key: No key defined

Min footing T&S reinf Area 0.79 in2
Min footing T&S reinf Area per foot 0.22 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 11.11 in	#4@ 22.22 in
#5@ 17.22 in	#5@ 34.44 in
#6@ 24.44 in	#6@ 48.89 in

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,215.3	2.78	3,375.8
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	1,215.3	O.T.M. =	3,375.8

Resisting/Overturning Ratio = **1.55**
Vertical Loads used for Soil Pressure = 2,028.8 lbs

.....RESISTING.....

Item		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		624.7	3.33	2,082.0
Soil Over HL (bel. water tbl)			3.33	2,082.0
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	145.8	1.17	170.1
Surcharge Over Toe	=			
Stem Weight(s)	=	800.0	2.67	2,133.1
Earth @ Stem Transitions	=			
Footing Weight	=	458.3	1.83	840.0
Key Weight	=		2.92	
Vert. Component	=			
Total =		2,028.8 lbs	R.M.=	5,225.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.090 in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

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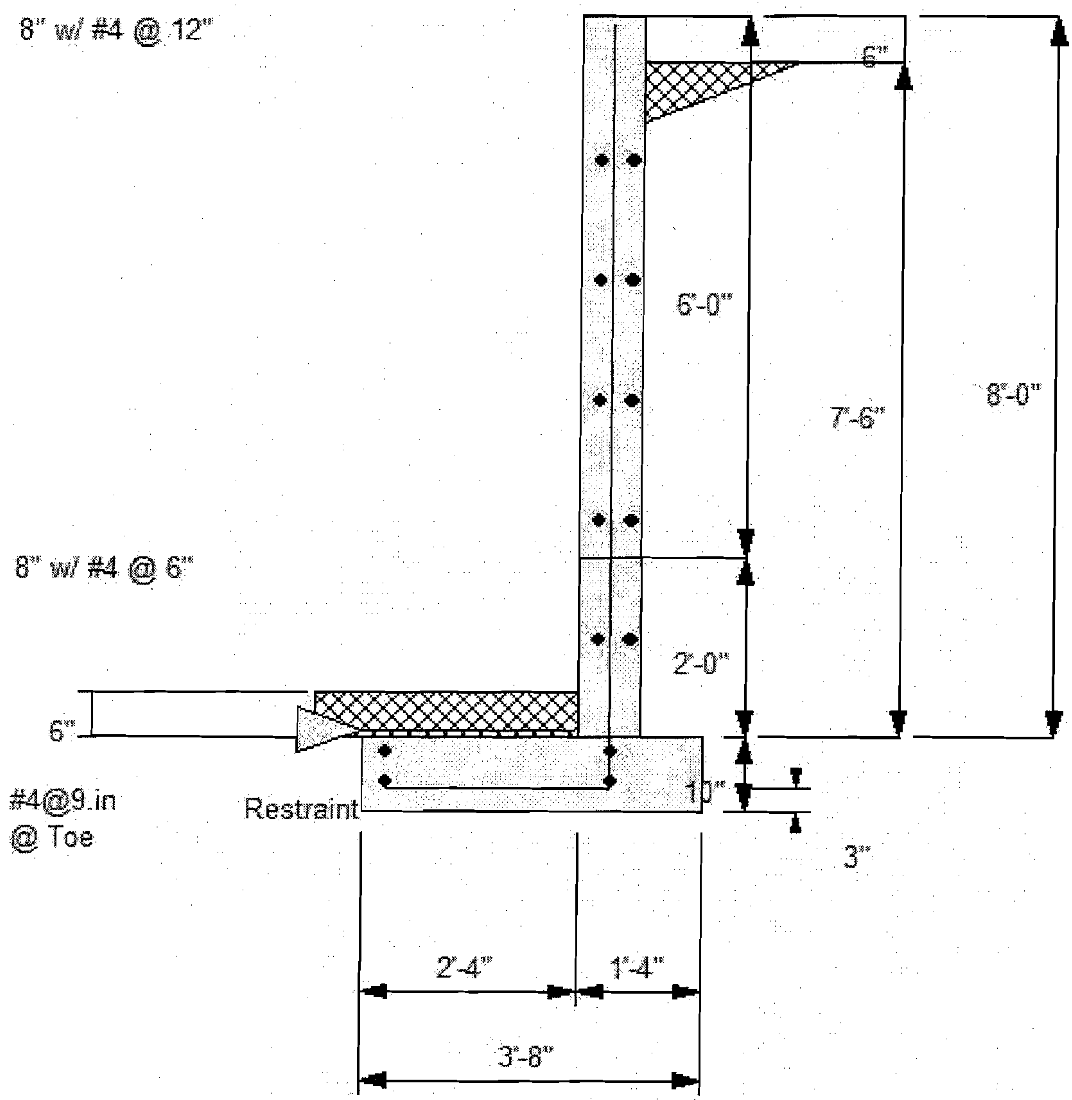
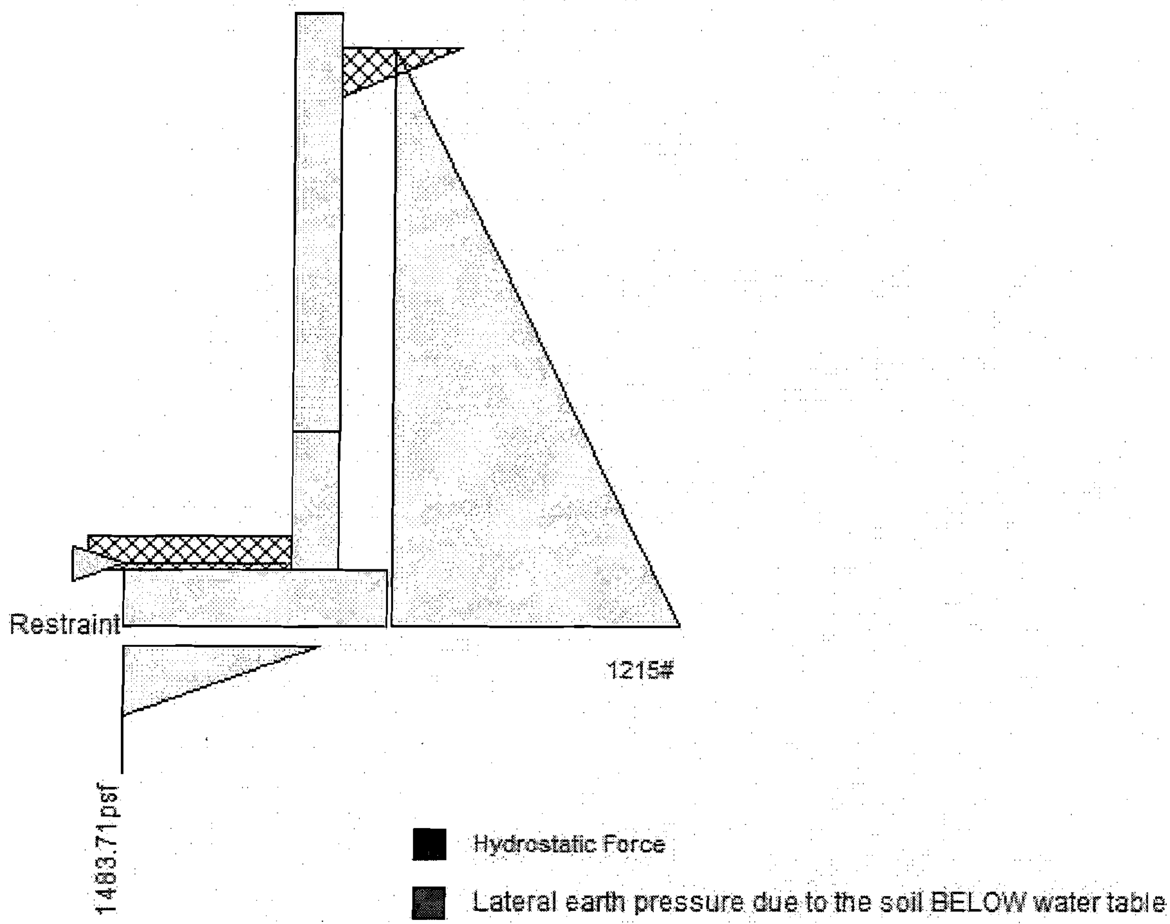
Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16




Restraint
1215#
1483.71psf
Hydrostatic Force
Lateral earth pressure due to the soil BELOW water table

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Cantilevered Retaining Wall

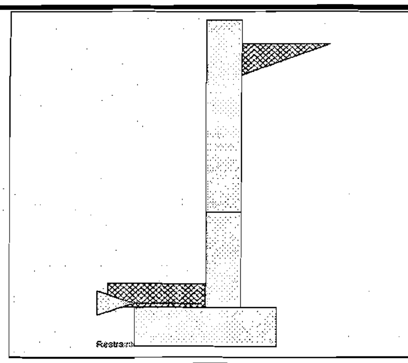
Code: IBC 2018,ACI 318-14,TMS 402-16

Criteria

Retained Height	=	5.50 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	125.00 pcf
Soil Density, Toe	=	125.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	1.73 OK
Slab Resists All Sliding !		
Total Bearing Load	=	1,475 lbs
...resultant ecc.	=	7.16 in
Soil Pressure @ Toe	=	1,335 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,868 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	10.3 psi OK
Footing Shear @ Heel	=	7.7 psi OK
Allowable	=	75.0 psi

Sliding Calcs

Lateral Sliding Force	=	701.9 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2018,ACI
Dead Load	1.400
Live Load	1.700
Earth, H	1.700
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	2.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 4	# 4
Rebar Spacing	=	12.00	10.00
Rebar Placed at	=	Center	Center
Design Data			
fb/FB + fa/Fa	=	**0.125**	**0.411**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	364.4	899.9
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	425.2	1,649.9
Moment.....Allowable	ft-# =	3,387.6	4,014.1
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	7.6	18.7
Shear.....Allowable	psi =	75.0	75.0
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	4.00	4.00
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0257 in2/ft		
(4/3) * As :	0.0342 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0996 in2/ft		
(4/3) * As :	0.1328 in2/ft	Min Stem T&S Reinf Area 0.384 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.24 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.33 ft
Heel Width	=	1.33
Total Footing Width	=	2.67
Footing Thickness	=	10.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	1.67 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,868	0 psf
Mu' : Upward	=	15,914	1 ft-#
Mu' : Downward	=	2,799	253 ft-#
Mu: Design	=	1,093	251 ft-#
Actual 1-Way Shear	=	10.33	7.70 psi
Allow 1-Way Shear	=	40.00	40.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	
Footing Torsion, Tu	=		0.00 ft-lbs
Footing Allow. Torsion, phi Tu	=		0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: phiMn = phi'5'lambda'sqrt(fc)'Sm
Key: No key defined

Min footing T&S reinf Area	0.58	in2
Min footing T&S reinf Area per foot	0.22	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 11.11 in	#4@ 22.22 in
#5@ 17.22 in	#5@ 34.44 in
#6@ 24.44 in	#6@ 48.89 in

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING..... Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		701.9	2.11	1,481.9
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	701.9	O.T.M. =	1,481.9

Resisting/Overturning Ratio = **1.73**
Vertical Loads used for Soil Pressure = 1,474.7 lbs

.....RESISTING.....		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		458.1	2.33	1,068.7
Soil Over HL (bel. water tbl)			2.33	1,068.7
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	83.3	0.67	55.5
Surcharge Over Toe	=			
Stem Weight(s)	=	600.0	1.67	999.8
Earth @ Stem Transitions	=			
Footing Weight	=	333.3	1.33	444.2
Key Weight	=		1.67	
Vert. Component	=			
Total =		1,474.7 lbs	**R.M.=**	2,568.2

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.083	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe. because the wall would then tend to rotate into the retained soil.

Mark Myers, P.E.
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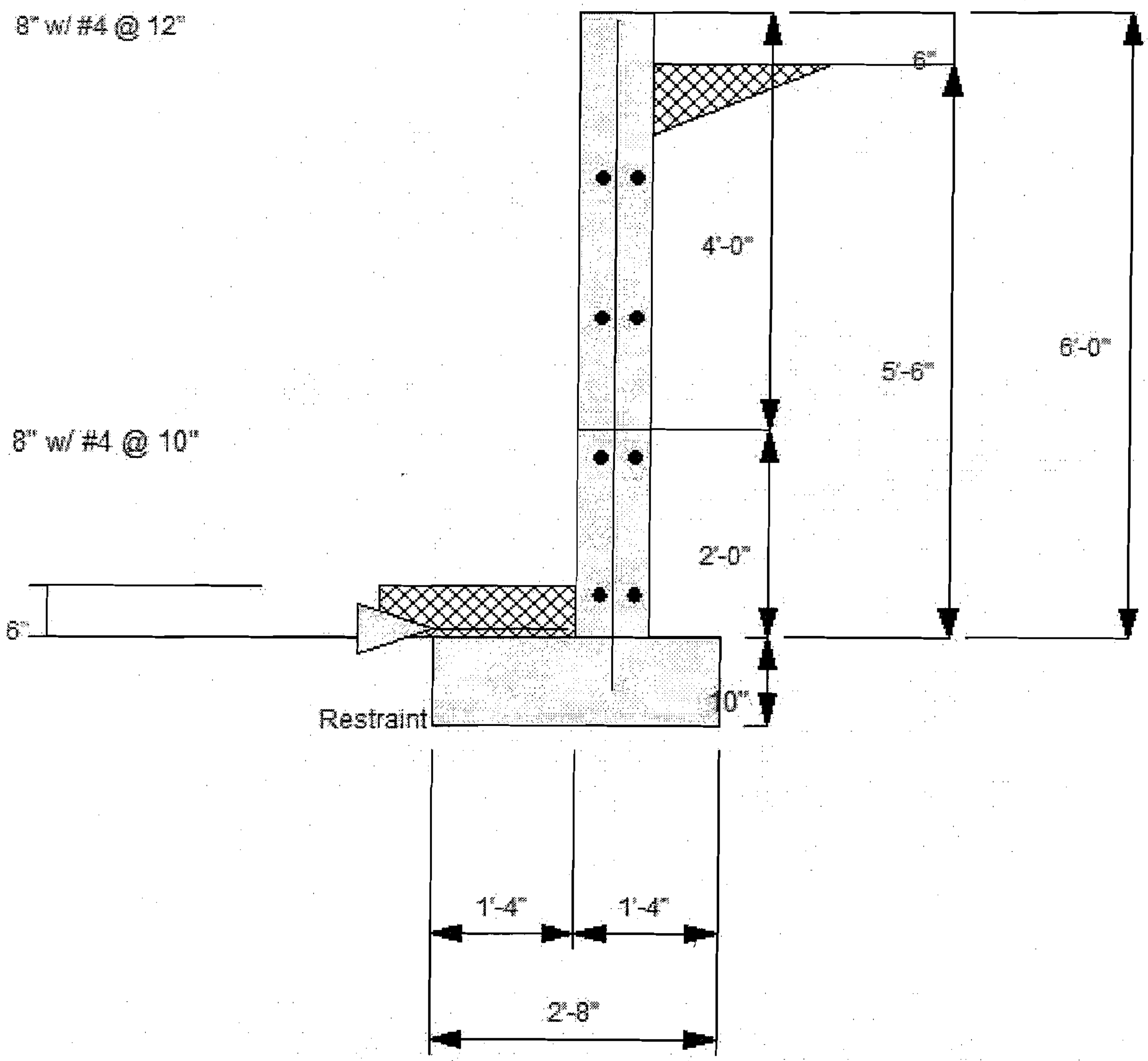
Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16



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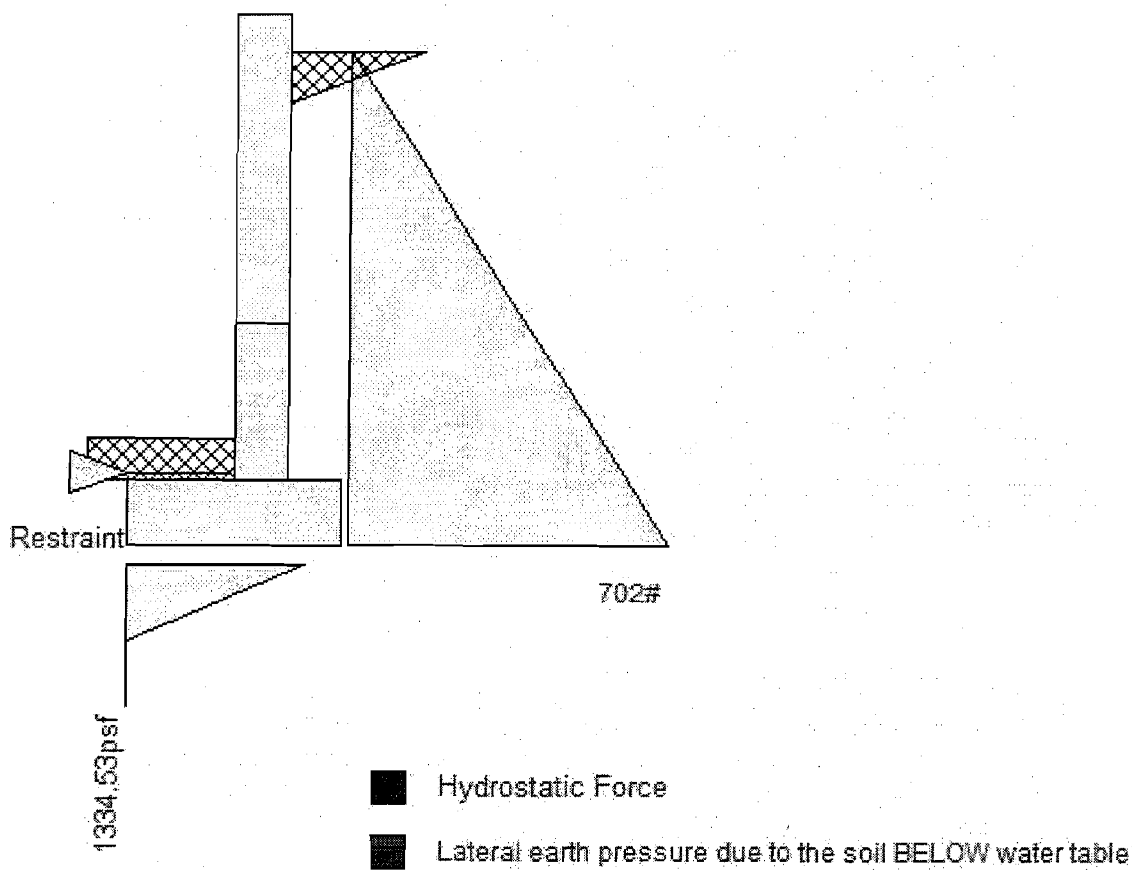

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Cantilevered Retaining Wall

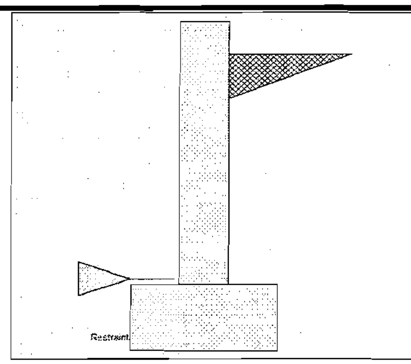
Code: IBC 2018,ACI 318-14,TMS 402-16

Criteria

Retained Height	=	3.50 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	125.00 pcf
Soil Density, Toe	=	125.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	2.23 OK

Slab Resists All Sliding !

Total Bearing Load	=	992 lbs
...resultant ecc.	=	4.08 in
Soil Pressure @ Toe	=	1,001 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,402 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.1 psi OK
Footing Shear @ Heel	=	3.3 psi OK
Allowable	=	75.0 psi

Sliding Calcs

Lateral Sliding Force	=	354.4 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2018,ACI
Dead Load	1.400
Live Load	1.700
Earth, H	1.700
Wind, W	1.000
Seismic, E	1.000

Stem Construction

			Bottom
			Stem OK
Design Height Above Ftg	ft	=	0.00
Wall Material Above "Ht"		=	Concrete
Design Method		=	LRFD
Thickness		=	8.00
Rebar Size		=	# 4
Rebar Spacing		=	10.00
Rebar Placed at		=	Center
Design Data			
fb/FB + fa/Fa		=	**0.105**
Total Force @ Section			
Service Level	lbs	=	
Strength Level	lbs	=	364.4
Moment....Actual			
Service Level	ft-#	=	
Strength Level	ft-#	=	425.2
Moment.....Allowable		=	4,014.1
Shear.....Actual			
Service Level	psi	=	
Strength Level	psi	=	7.6
Shear.....Allowable	psi	=	75.0
Anet (Masonry)	in2	=	
Rebar Depth 'd'	in	=	4.00
Masonry Data			
f'm	psi	=	
Fs	psi	=	
Solid Grouting		=	
Modular Ratio 'n'		=	
Wall Weight	psf	=	100.0
Short Term Factor		=	
Equiv. Solid Thick.		=	
Masonry Block Type		=	Medium Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	2,500.0
Fy	psi	=	60,000.0

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Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0257 in2/ft		
(4/3) * As :	0.0342 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.24 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	0.67 ft
Heel Width	=	1.33
Total Footing Width	=	2.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	1.67 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,402	0 psf
Mu' : Upward	=	3,322	32 ft-#
Mu' : Downward	=	561	183 ft-#
Mu: Design	=	230	151 ft-#
Actual 1-Way Shear	=	0.07	3.33 psi
Allow 1-Way Shear	=	40.00	40.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: phiMn = phi'5'lambda'sqrt(fc)'Sm
Key: No key defined

Min footing T&S reinf Area 0.52 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		354.4	1.50	531.6
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	354.4	**O.T.M.** =	531.6

Resisting/Overturning Ratio = **2.23**
Vertical Loads used for Soil Pressure = 991.5 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		291.5	1.67	485.9
Soil Over HL (bel. water tbl)			1.67	485.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=			
Surcharge Over Toe	=			
Stem Weight(s)	=	400.0	1.00	400.1
Earth @ Stem Transitions	=			
Footing Weight	=	300.0	1.00	300.0
Key Weight	=		1.67	
Vert. Component	=			
Total =		991.5 lbs	**R.M.=**	1,186.0

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.056	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

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Cantilevered Retaining Wall

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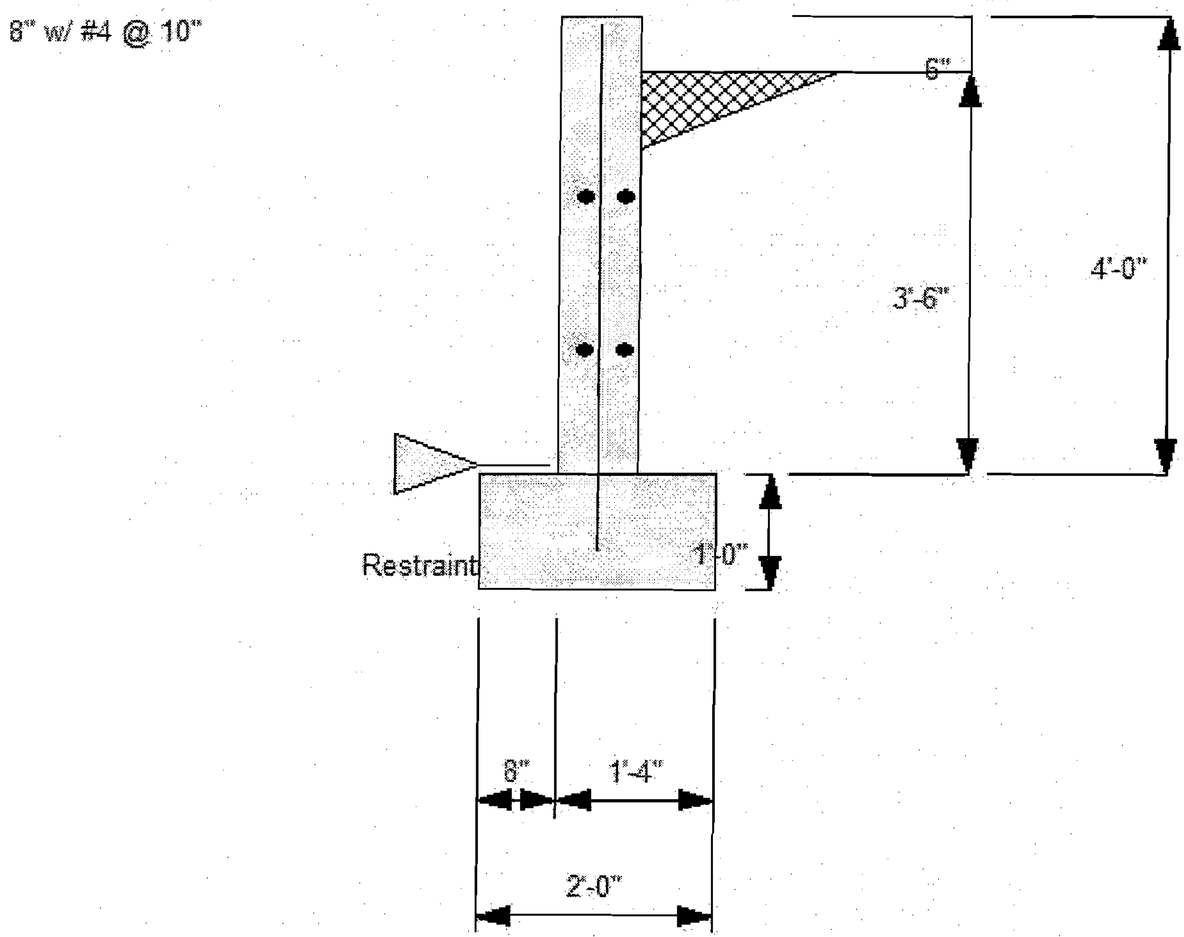

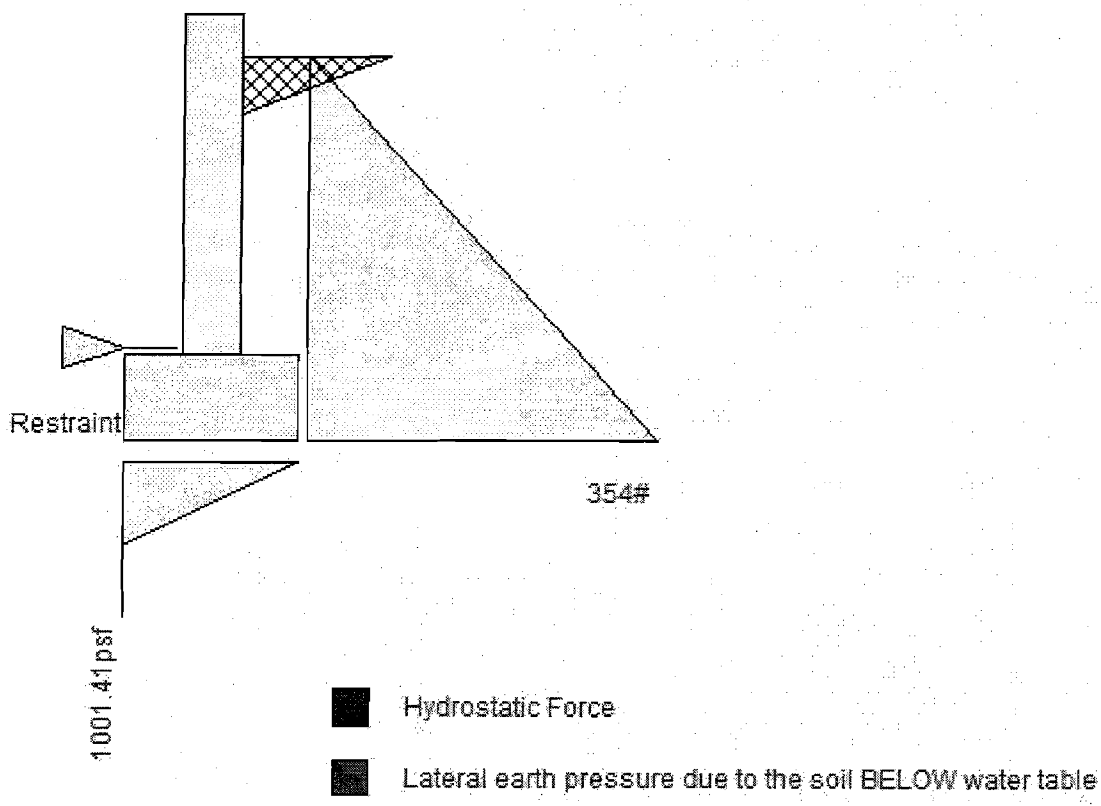
Restraint
354#
1001.41psf
Hydrostatic Force
Lateral earth pressure due to the soil BELOW water table