MYERS ENGINEERING

LATERAL ANALYSIS & GRAVITY CALCULATIONS

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Mark Myers, PE
Date: 2020.08.31
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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: Marbella Residence
7311 West Mercer Way
Mercer Island, WA 98332

August 31, 2020

2015 INTERNATIONAL BUILDING CODE
110 MPH WIND, EXPOSURE C, K_{zt} = 1.00
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

3206 50th Street Court NW, Suite 210-B
Gig Harbor, WA 98335
Phone: 253-858-3248
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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.--------HF#2
BEAMS OR HEADERS 4X - 6X OR LARGER--------DF#2
LEDGERS AND TOP PLATES--------HF#2
STUDS 2X4 OR 2X6--------HF Stud
POSTS
4X4--------DF#2
4X6--------DF#2
6X6--------DF#1

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

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

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

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

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

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

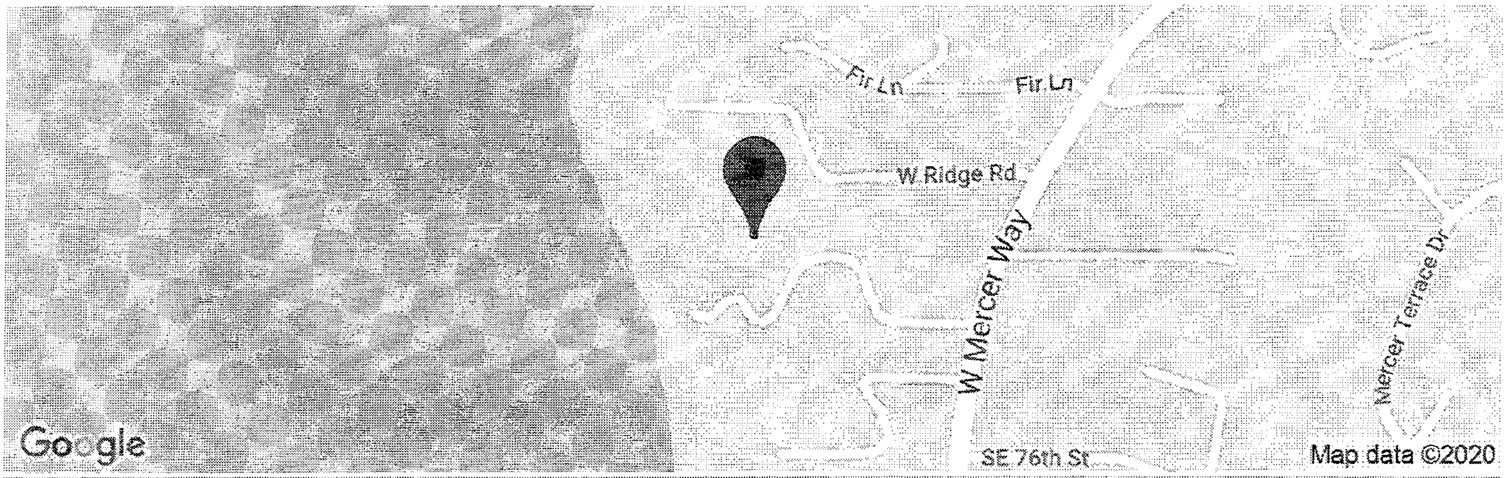
ENGINEERED I-JOISTS
-FLOOR JOISTS & BEAMS OF EQUAL OR BETTER CAPACITY MAY BE SUBSTITUTED FOR THOSE SHOWN ON THIS PLAN, "EQUAL" IS DEFINED AS HAVING MOMENT CAPACITY, SHEAR CAPACITY, AND STIFFNESS WITHIN 3% OF THE SPECIFIED JOISTS OR BEAMS.

OSHPD

Marbella

7275 W Mercer Way, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.5367172, -122.242623



Date	8/7/2020, 2:00:46 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.472	MCE_R ground motion. (for 0.2 second period)
S_1	0.562	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.472	Site-modified spectral acceleration value
S_{M1}	0.843	Site-modified spectral acceleration value
S_{DS}	0.981	Numeric seismic design value at 0.2 second SA
S_{D1}	0.562	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.613	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.613	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.472	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.559	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.624	Factored deterministic acceleration value. (0.2 second)
S1RT	0.562	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.608	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.303	Factored deterministic acceleration value. (1.0 second)
PGAd	1.37	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.944	Mapped value of the risk coefficient at short periods
C_{R1}	0.925	Mapped value of the risk coefficient at a period of 1 s

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PROJECT : **Marbella Residence**

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LATERAL ANALYSIS :

BASED ON 2015 INTERNATIONAL BUILDING CODE (IBC)

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

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

SEISMIC DESIGN:

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

Seismic Design Data:

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

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

$S_s := 1.472$ $\quad S_1 := 0.562$ $\quad S_{ms} := 1.472$ $\quad S_{m1} := 0.843$

Equation 16-39 $\quad S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.98$ $\quad$ Equation 16-40 $\quad S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.56$

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

Roof Slope Adjustment Factor: $\quad S_a := \dfrac{1}{\cos\left(\operatorname{atan}\left(\frac{3.5}{12}\right)\right)} = 1.04$

Plan Area for Each Level:

$A_1 := 1698\text{ft}^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1512\text{ft}^2$ (Framed Floor) $\quad A_{2b} := 1938\text{ft}^2 \cdot S_a$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(42\text{ft}) + 2(50\text{ft})$ (Main Floor) $\quad P_2 := 2(106\text{ft}) + 2(44\text{ft})$ (Lower Floor)

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

Weight of Structure at Each Level:

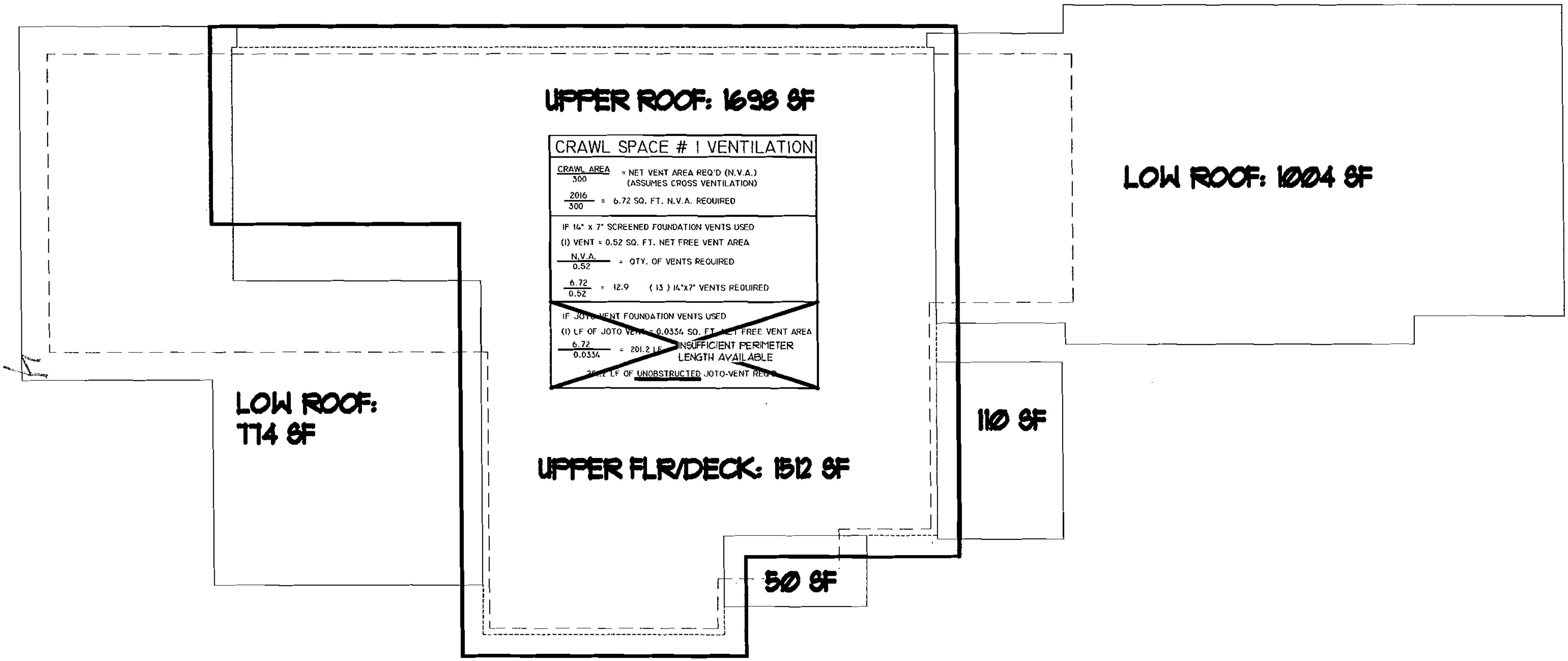
Story Weight at Upper Floor:

$$w_1 := 15 \cdot \text{psf} \cdot A_1 + 12 \cdot \text{psf} \cdot 4 \cdot \text{ft} \cdot P_1 = 35363.25\,\text{lb}$$

Story Weight at Main Floor:

$$w_2 := 15 \cdot \text{psf} \cdot A_{2a} + 15\text{psf} \cdot A_{2b} + 12 \cdot \text{psf} \cdot (4 \cdot \text{ft} \cdot P_1 + 4.5\text{ft} \cdot P_2) = 77993.25\,\text{lb}$$

$$W := w_1 + w_2 = 113356.5\,\text{lb}$$


UPPER ROOF: 1698 SF
CRAWL SPACE # 1 VENTILATION
CRAWL AREA / 300 = NET VENT AREA REQ'D (N.V.A.)
(ASSUMES CROSS VENTILATION)
2016 / 300 = 6.72 SQ. FT. N.V.A. REQUIRED
IF 14" x 7" SCREENED FOUNDATION VENTS USED
(1) VENT = 0.52 SQ. FT. NET FREE VENT AREA
N.V.A. / 0.52 = QTY. OF VENTS REQUIRED
6.72 / 0.52 = 12.9 (13) 14"x7" VENTS REQUIRED
~~IF JOTO-VENT FOUNDATION VENTS USED~~
~~(1) LF OF JOTO VENT = 0.0334 SQ. FT. NET FREE VENT AREA~~
~~6.72 / 0.0334 = 201.2 LF~~
INSUFFICIENT PERIMETER
LENGTH AVAILABLE
~~201.2 LF OF UNOBSTRUCTED JOTO-VENT REQ'D~~
UPPER FLR/DECK: 1512 SF
LOW ROOF: 1004 SF
LOW ROOF:
774 SF
110 SF
50 SF

Approximate Fundamental Period, $T_{a:}$

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE7-10 Table 12.8-2) $\quad h_n := 20$ (Structural Height per ASCE7-10 Sect. 11.2)

$T_a := C_t \cdot h_n^{\chi} = 0.19$ (ASCE7-10 Eq. 12.8-7) $\quad T_L := 6$ (per ASCE7-10 Fig. 22-12)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.46$ (ASCE7-10 Eq. 12.8-3)

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

$C_s := \dfrac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.15$ Total Base Shear: $V_E := C_s \cdot W = 17113.92\,\text{lb}$

Vertical Shear distribution at each level:

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

$h_1 := 18\text{ft}$ $\quad h_2 := 10\text{ft}$ (Height from base to level x)

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.45$ $\quad F_1 := C_{v1} \cdot V_E = 7690.71\,\text{lb}$ Story Shear at Upper Floor

$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.55$ $\quad F_2 := C_{v2} \cdot V_E = 9423.21\,\text{lb}$ Story Shear at Main Floor

WIND DESIGN

Use analytical procedure of ASCE 7-10 Chapter 27 (Directional Procedure for buildings of all heights)

$V := 110$ 3-Sec Peak Gust (MPH) for Risk Category II (Figure 26.5-1A).

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 20 \cdot ft$ Mean Roof Height as per Sect. 26.2

Exposure Category C (ASCE7-10 Sect. 26.7.3)

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

$x := 1ft$ $H := 1ft$ $L_h := 1ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.85\left(\frac{H}{L_h}\right) = 0.85 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.75 \qquad K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0 \qquad K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1$$

$G := 0.85$ Gust Effect Factor (ASCE7-10 Sect. 26.9.1)

Building is an Enclosed Building as per ASCE7-10 Sect. 26.10

$GC_{pi} := .18$ +/- Internal Pressure Coefficients (ASCE7-10 Table 26.11-1)

Velocity Pressure Exposure Coefficient (Table 27.3-1):

$z_g := 900ft$ $\alpha := 9.5$ (per ASCE7-10 Table 26.9-1 based on Exposure Category)
z_g=1200ft, α=7.0 (Exp B), z_g=900ft, α=9.5 (Exp C), z_g=700ft, α=11.5 (Exp D)

$z_1 := 20ft$ $z_2 := 15ft$ Height from ground to level x (z_{min} = 15ft)

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

External Pressure Coefficients w/ Roof Pitch = 3.5/12 (16 degrees) Front to Back & 3.5/12 (16 degrees) Side to Side Taken from Figure 27.4-1

Front to Back:

$L_{fb} := 44ft$ $B_{fb} := 106ft$ $\frac{L_{fb}}{B_{fb}} = 0.42$ $\frac{h}{L_{fb}} = 0.45$

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

$C_{pf2} := -0.11$ Windward Roof

$C_{pf3} := -.52$ Leeward Roof

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

Side to Side:

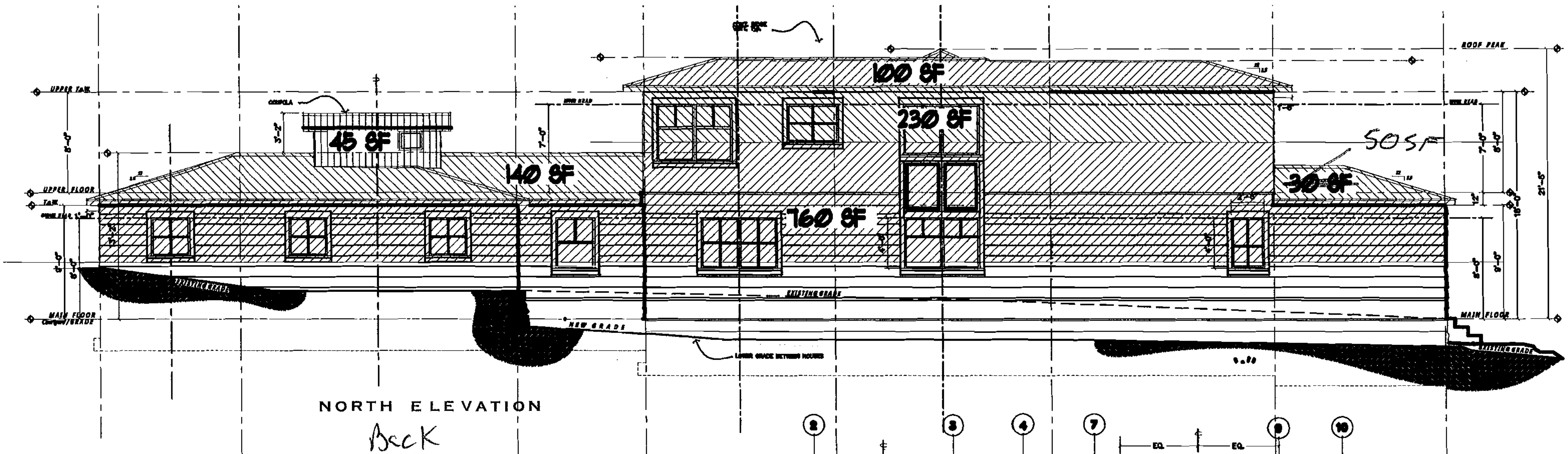
$L_{ss} := 106ft$ $B_{ss} := 44ft$ $\frac{L_{ss}}{B_{ss}} = 2.41$ $\frac{h}{L_{ss}} = 0.19$

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

$C_{ps2} := 0.04$ Windward Roof

$C_{ps3} := -.52$ Leeward Roof

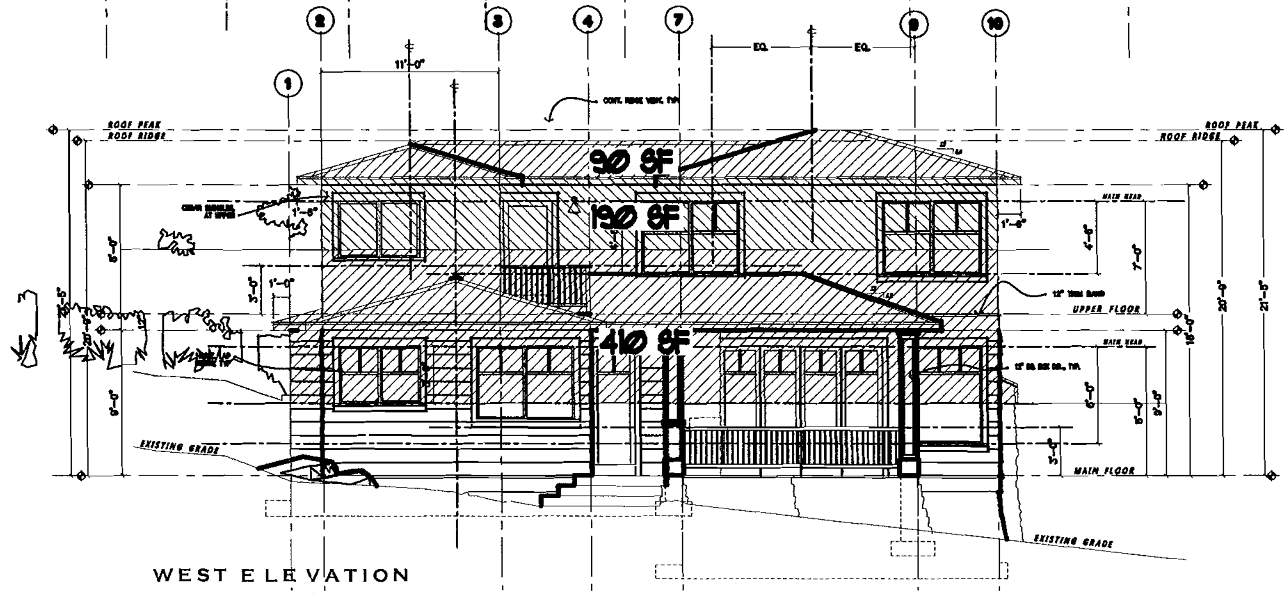
$C_{ps4} := -.28$ Leeward Wall



NORTH ELEVATION

Back

√



WEST ELEVATION

Left

Velocity Pressure (q_z) Evaluated at Height (z) (Equation 23.3-1)

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot V^2 = 23.75$ $\quad q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot V^2 = 22.35$ $\quad q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 = 23.75$

Design Wind Pressures p = qGC_p - $q_i(GC_{pi})$ (Equation 27.4-1) where q_i will conservatively be taken equal to q_h

Windward Wall Both Directions $\quad p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 16.15\, lb \cdot ft^{-2}$ $\quad p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 15.2\, lb \cdot ft^{-2}$

Windward Roof Front to Back $\quad p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -2.22\, lb \cdot ft^{-2}$

Leeward Roof Front to Back $\quad p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -10.5\, lb \cdot ft^{-2}$

Leeward Wall Front to Back $\quad p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -10.09\, lb \cdot ft^{-2}$

Windward Roof Side to Side $\quad p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 0.81\, lb \cdot ft^{-2}$

Leeward Roof Side to Side $\quad p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -10.5\, lb \cdot ft^{-2}$

Leeward Wall Side to Side $\quad p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -5.65\, lb \cdot ft^{-2}$

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:

$p_{wr1} - p_{lr1} = 8.28\, lb \cdot ft^{-2}$ $\quad p_{ww1} - p_{lw1} = 26.24\, lb \cdot ft^{-2}$ $\quad p_{ww2} - p_{lw1} = 25.29\, lb \cdot ft^{-2}$

$p_{wr2} - p_{lr2} = 11.3\, lb \cdot ft^{-2}$ $\quad p_{ww1} - p_{lw2} = 21.8\, lb \cdot ft^{-2}$ $\quad p_{ww2} - p_{lw2} = 20.85\, lb \cdot ft^{-2}$

<u>Wind Pressure at Upper Roof (Front to Back):</u>

$$V_{1W} := (p_{wr1} - p_{lr1})100ft^2 + (p_{ww1} - p_{lw1}) \cdot 230 \cdot ft^2 = 6862.67\, lb$$

<u>Wind Pressure at Main Floor (Front to Back):</u>

$$V_{2W} := (p_{wr1} - p_{lr1})190ft^2 + (p_{ww2} - p_{lw1}) \cdot 805 \cdot ft^2 = 21931.37\, lb$$

<u>Wind Pressure at Upper Roof (Side to Side):</u>

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 90ft^2 + (p_{ww1} - p_{lw2}) \cdot 190ft^2 = 5159.12\, lb$$

<u>Wind Pressure at Main Floor (Side to Side):</u>

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 410ft^2 = 8548.56\, lb$$

Determine Component & Cladding loads:

Design Wind Pressures $p = q_h[(GC_p) - (GC_{pi})]$ (Equation 30.4-1)

(GC_p) is given in Figures 30.4-1 thru 30.4-7

(GC_{pi}) is given in Table 26.11-1 (See above)

$GC_{p1in} := 0.5$ $\quad GC_{p2in} := 0.5$ $\quad GC_{p3in} := 0.5$ $\quad$ Figure 30.4-2B (θ = 16 degrees)

$GC_{p1out} := -0.9$ $\quad GC_{p2out} := -1.7$ $\quad GC_{p3out} := -2.6$ $\quad GC_{p2oh} := -2.2$ $\quad GC_{p3oh} := -3.7$

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

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

$p_1 := q_h \cdot \left[\left(GC_{p1out}\right) - \left(GC_{pi}\right)\right] psf$ $\quad p_1 = -25.65\,lb \cdot ft^{-2}$ (Zone 1)

$p_2 := q_h \cdot \left[\left(GC_{p2out}\right) - \left(GC_{pi}\right)\right] psf$ $\quad p_2 = -44.64\,lb \cdot ft^{-2}$ (Zone 2)

$p_3 := q_h \cdot \left[\left(GC_{p3out}\right) - \left(GC_{pi}\right)\right] psf$ $\quad p_3 = -66.01\,lb \cdot ft^{-2}$ (Zone 3)

$p_2 := q_h \cdot \left(\left(GC_{p2oh}\right)\right) psf$ $\quad p_2 = -52.24\,lb \cdot ft^{-2}$ (Zone 2 Overhang)

$p_3 := q_h \cdot \left(\left(GC_{p3oh}\right)\right) psf$ $\quad p_3 = -87.86\,lb \cdot ft^{-2}$ (Zone 3 Overhang)

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

$p_4 := q_h \cdot \left[\left(GC_{p4out}\right) - \left(GC_{pi}\right)\right] psf$ $\quad p_4 = -30.4\,lb \cdot ft^{-2}$ (Zone 4)

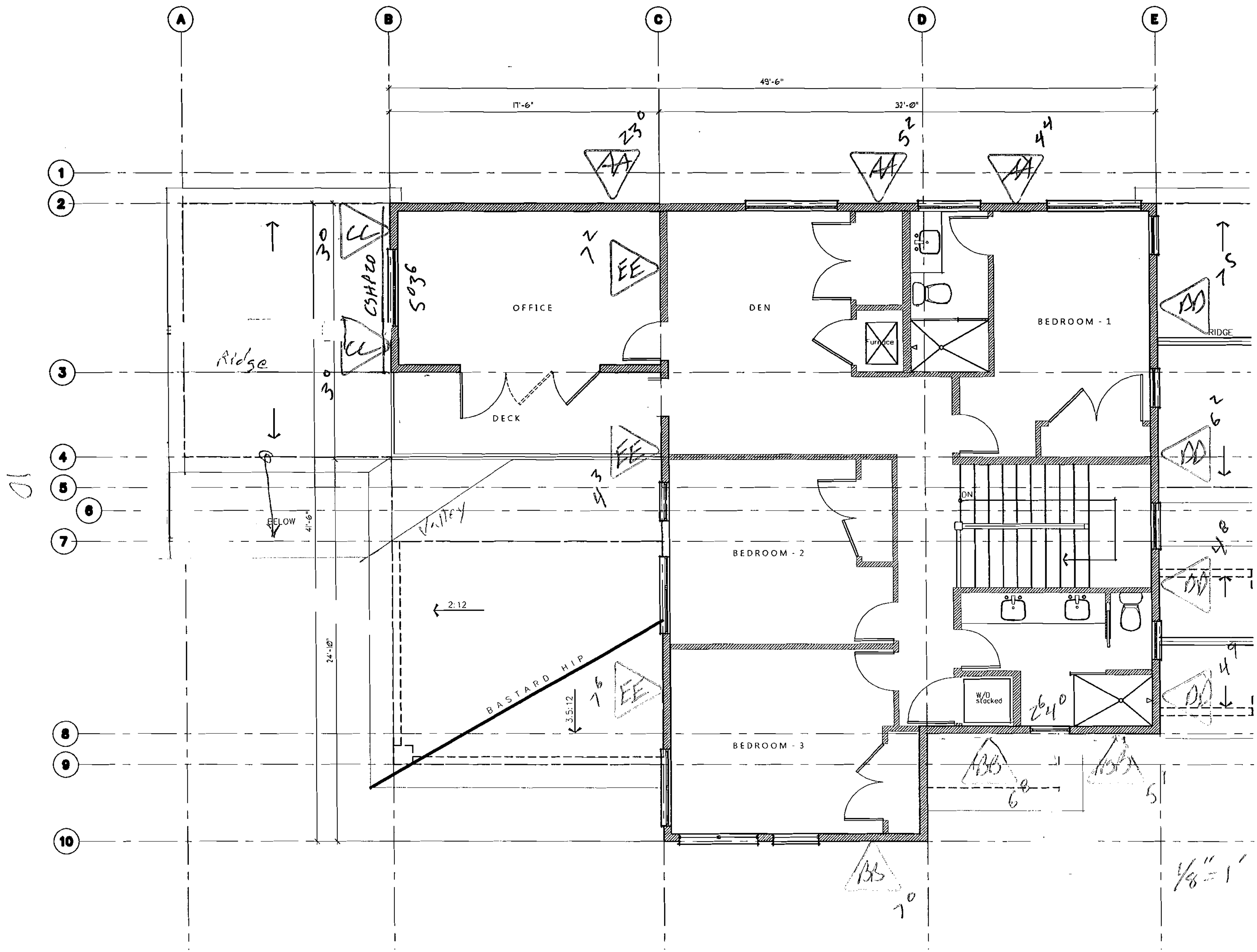
$p_5 := q_h \cdot \left[\left(GC_{p5out}\right) - \left(GC_{pi}\right)\right] psf$ $\quad p_5 = -37.52\,lb \cdot ft^{-2}$ (Zone 5)

Net pressure shall not be less than 16 psf for Components and Cladding (ASCE 7-10 Sec. 30.2.2)

a = 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than 4% of least horizontal dimension or 3ft

$0.1(44ft) = 4.4\,ft$ $\quad 0.4 \cdot h = 8\,ft$ $\quad 0.04(44ft) = 1.76\,ft$

Therefore $\quad a := 4.4ft$



49'-6"
17'-6"
32'-0"
OFFICE
DEN
BEDROOM - 1
DECK
RIDGE
BEDROOM - 2
BEDROOM - 3
Furnace
W/D stacked
DN
2:12
BASTARD HIP
3.5:12
BELOW
41'-6"
24'-10"
Ridge
Valley
5036
CSHP20
1/8" = 1'

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WALL AA:

Story Shear due to Wind: $V_{3W} = 5159.12\,lb$ Story Shear due to Seismic: $F_1 = 7690.71\,lb$

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

Shear Wall Length: $Laa_w := (23 + 5.17 + 4.33)ft = 32.5\,ft$ $Laa_s := (23 + 5.17 + 4.33)ft = 32.5\,ft$

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

$vaa = 47.62\,lb \cdot ft^{-1}$ $\dfrac{vaa}{C_o} = 47.62\,lb \cdot ft^{-1}$ $E_{aa} = 82.82\,lb \cdot ft^{-1}$ $\dfrac{E_{aa}}{C_o} = 82.82\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{aa} := 4.33 \cdot ft$ Plate Height: $Pt := 8 \cdot ft$

$W_{aa} := (15 \cdot psf) \cdot 9 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRaa := \dfrac{W_{aa} \cdot L_{aa}}{2}$ $DLRaa = 465.47\,lb$

Chord Force:

$CFaa_w := \dfrac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_w = 380.98\,lb$ $CFaa_s := \dfrac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_s = 662.58\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 101.7\,lb$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 447.25\,lb$

No Holdowns Required

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

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

$B_p := \dfrac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 3.43\,ft$ $\dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 1.97\,ft$

16d @ 16" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vaa} = 28.89\,ft$ $\dfrac{(Z_B \cdot C_o)}{E_{aa}} = 16.61\,ft$

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

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WALL BB:

Story Shear due to Wind: $V_{3W} = 5159.12\,\text{lb}$

Story Shear due to Seismic: $F_1 = 7690.71\,\text{lb}$

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

Distance between shear walls: $L_1 := 42\cdot\text{ft}$

Shear Wall Length: $Lbb_w := (7 + 6.67 + 5.75)\text{ft} = 19.42\,\text{ft}$ $\quad Lbb_s := (7 + 6.67 + 5.75)\text{ft} = 19.42\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{12.42\cdot\text{ft}}{15\cdot\text{ft}}\right)\cdot 100 \quad \% = 82.8$

Max Opening Height = 4ft-0in, Therefore $C_o := 0.91$
per AF&PA SDPWS Table 4.3.3.5

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

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

$vbb = 79.7\,\text{lb}\cdot\text{ft}^{-1} \qquad \dfrac{vbb}{C_o} = 87.58\,\text{lb}\cdot\text{ft}^{-1}$

$E_{bb} = 138.61\,\text{lb}\cdot\text{ft}^{-1} \qquad \dfrac{E_{bb}}{C_o} = 152.32\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{bb} := 7\cdot\text{ft}$ $\quad$ Plate Height: $Pt := 8\cdot\text{ft}$

$W_{bb} := (15\cdot\text{psf})\cdot 4\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$

$DLRbb := \dfrac{W_{bb}\cdot L_{bb}}{2} \qquad DLRbb = 490\,\text{lb}$

Chord Force:

$CFbb_w := \dfrac{vbb\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}} \qquad CFbb_w = 700.64\,\text{lb}$

$CFbb_s := \dfrac{E_{bb}\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}} \qquad CFbb_s = 1218.52\,\text{lb}$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6\cdot DLRbb = 406.64\,\text{lb}$

$HDFbb_s := CFbb_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRbb = 991.84\,\text{lb}$

No Holdowns Required

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

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

$B_n := \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{vbb} = 1.86\,\text{ft} \qquad \dfrac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{bb}} = 1.07\,\text{ft}$

16d @ 12" o.c.

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

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

$As := \dfrac{\left(Z_B\cdot C_o\right)}{vbb} = 15.71\,\text{ft} \qquad \dfrac{\left(Z_B\cdot C_o\right)}{E_{bb}} = 9.03\,\text{ft}$

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

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WALL CC:

Story Shear due to Wind: $V_{1W} = 6862.67\,\text{lb}$ Story Shear due to Seismic: $F_1 = 7690.71\,\text{lb}$

Bldg Width in direction of Load: $L_t := 49.5\cdot\text{ft}$ Distance between shear walls: $L_1 := 17.5\cdot\text{ft}$

Shear Wall Length: $Lcc_w := (2\cdot3)\text{ft} = 6\,\text{ft}$ $Lcc_s := (2\cdot3)\text{ft} = 6\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{10\cdot\text{ft}}{10\cdot\text{ft}}\right)\cdot100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vcc := \dfrac{\dfrac{0.6V_{1W}}{L_t}\cdot\dfrac{L_1}{2}}{Lcc_w}$ Seismic Force: $\rho := 1.0$ $E_{cc} := \dfrac{\rho\cdot\dfrac{0.7F_1}{L_t}\cdot\dfrac{L_1}{2}}{Lcc_s}$

$vcc = 121.31\,\text{lb}\cdot\text{ft}^{-1}$ $\dfrac{vcc}{C_o} = 121.31\,\text{lb}\cdot\text{ft}^{-1}$ $E_{cc} = 158.6\,\text{lb}\cdot\text{ft}^{-1}$ $\dfrac{E_{cc}}{C_o} = 158.6\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{cc} := 11\cdot\text{ft}$ Plate Height: $Pt := 8\cdot\text{ft}$

$W_{cc} := (15\cdot\text{psf})\cdot3\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot0\text{ft}$ $DLRcc := \dfrac{W_{cc}\cdot L_{cc}}{2}$ $DLRcc = 687.5\,\text{lb}$

Chord Force:

$CFcc_w := \dfrac{vcc\cdot6\text{ft}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_w = 529.35\,\text{lb}$ $CFcc_s := \dfrac{E_{cc}\cdot6\text{ft}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_s = 692.09\,\text{lb}$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 116.85\,\text{lb}$ $HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS})\cdot DLRcc = 374.05\,\text{lb}$

No Holdown Required, provide CSHP20 straps top & bottom of opening

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

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

$B_P := \dfrac{(C_D\cdot Z_N\cdot C_o)}{vcc} = 1.35\,\text{ft}$ $\dfrac{(C_D\cdot Z_N\cdot C_o)}{E_{cc}} = 1.03\,\text{ft}$

16d @ 12" o.c.

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

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

$AS := \dfrac{(Z_B\cdot C_o)}{vcc} = 11.34\,\text{ft}$ $\dfrac{(Z_B\cdot C_o)}{E_{cc}} = 8.68\,\text{ft}$

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

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WALL DD:

Story Shear due to Wind: $V_{1W} = 6862.67\,\text{lb}$ Story Shear due to Seismic: $F_1 = 7690.71\,\text{lb}$

Bldg Width in direction of Load: $L_t := 49.5\cdot\text{ft}$ Distance between shear walls: $L_1 := 32\cdot\text{ft}$

Shear Wall Length: $Ldd_w := (7.42 + 6.17 + 4.67 + 4.75)\text{ft} = 23.01\,\text{ft}$ $Ldd_s := (7.42 + 6.17 + 4.67 + 4.75)\text{ft} = 23.01\,\text{ft}$

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

$vdd = 57.84\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{vdd}{C_o} = 57.84\,\text{lb}\cdot\text{ft}^{-1}$ $E_{dd} = 75.62\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{E_{dd}}{C_o} = 75.62\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{dd} := 4.67\cdot\text{ft}$ Plate Height: $Pt := 8\cdot\text{ft}$

$W_{dd} := (15\cdot\text{psf})\cdot 3\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$ $DLRdd := \frac{W_{dd}\cdot L_{dd}}{2}$ $DLRdd = 291.88\,\text{lb}$

Chord Force:

$CFdd_w := \frac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_w = 462.74\,\text{lb}$ $CFdd_s := \frac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_s = 605\,\text{lb}$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 287.61\,\text{lb}$ $HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 469.97\,\text{lb}$

No Holdown Required

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vdd} = 2.82\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{dd}} = 2.16\,\text{ft}$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vdd} = 23.79\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_{dd}} = 18.2\,\text{ft}$

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

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WALL EE:

Story Shear due to Wind: $V_{1W} = 6862.67\,lb$ Story Shear due to Seismic: $F_1 = 7690.71\,lb$

Bldg Width in direction of Load: $L_t := 49.5\cdot ft$ Distance between shear walls: $L_1 := 32\cdot ft$ $L_2 := 17.5ft$

Shear Wall Length: $Lee_w := (7.5 + 4.25 + 7.17)ft = 18.92\,ft$ $Lee_s := (7.5 + 4.25 + 7.17)ft = 18.92\,ft$

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

$vee = 108.82\,lb\cdot ft^{-1}$ $\frac{vee}{C_o} = 108.82\,lb\cdot ft^{-1}$ $E_{ee} = 142.27\,lb\cdot ft^{-1}$ $\frac{E_{ee}}{C_o} = 142.27\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{ee} := 4.25\cdot ft$ Plate Height: $Pt := 8\cdot ft$

$W_{ee} := (15\cdot psf)\cdot 17\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRee := \frac{W_{ee}\cdot L_{ee}}{2}$ $DLRee = 711.87\,lb$

Chord Force:

$CFee_w := \frac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_w = 870.53\,lb$ $CFee_s := \frac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_s = 1138.16\,lb$

Holdown Force:

$HDFee_w := CFee_w - 0.6\cdot DLRee = 443.4\,lb$ $HDFee_s := CFee_s - (0.6 - 0.14S_{DS})DLRee = 808.84\,lb$

No Holdowns Required

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

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

$B_n := \frac{(Z_N\cdot C_D\cdot C_o)}{vee} = 1.5\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{ee}} = 1.15\,ft$

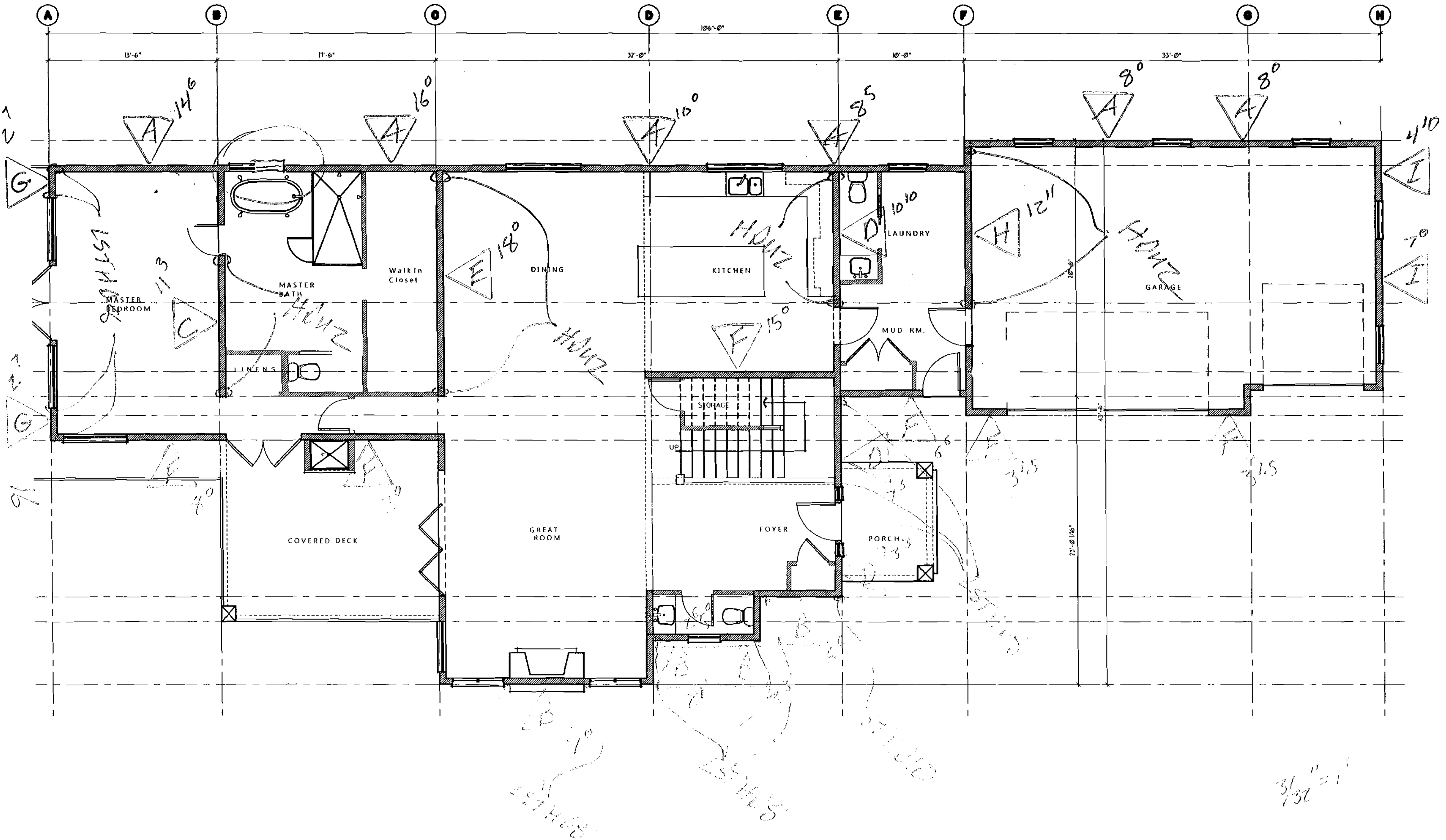
16d @ 12" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vee} = 12.65\,ft$ $\frac{(Z_B\cdot C_o)}{E_{ee}} = 9.67\,ft$

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


106'-0"
13'-6"
11'-6"
32'-0"
10'-0"
33'-0"
MASTER BEDROOM
MASTER BATH
Walk In Closet
LINENS
DINING
KITCHEN
LAUNDRY
MUD RM.
GARAGE
STORAGE
UP
COVERED DECK
GREAT ROOM
FOYER
PORCH
HOWZ
LSTHO

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WALL A:

Story Shear due to Wind: $V_{4W} = 8548.56\,lb$ Story Shear due to Seismic: $F_2 = 9423.21\,lb$

Bldg Width in direction of Load: $L_t := 43.5\cdot ft$ Distance between shear walls: $L_1 := 20.5\cdot ft$

Shear Wall Length: $La_w := (14.5 + 16 + 10 + 8.42 + 2\cdot 8)ft = 64.92\,ft$ $La_s := (14.5 + 16 + 10 + 8.42 + 2\cdot 8)ft = 64.92\,ft$

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

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

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

$va = 42.46\,lb\cdot ft^{-1}$ $\frac{va}{C_o} = 42.46\,lb\cdot ft^{-1}$ $E_a = 65.4\,lb\cdot ft^{-1}$ $\frac{E_a}{C_o} = 65.4\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_a := 8\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_a := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 8ft$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 680\,lb$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 382.12\,lb$ $CFa_w + CFaa_w = 763.1\,lb$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 588.64\,lb$ $CFa_s + CFaa_s = 1251.22\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = -25.88\,lb$ $HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 274.06\,lb$

No Holdowns Required

$HDFa_w + HDFaa_w = 75.81\,lb$ $HDFa_s + HDFaa_s = 721.31\,lb$

No Holdowns Required

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

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

$B_a := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 3.84\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 2.5\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{va} = 32.41\,ft$ $\frac{(Z_B\cdot C_o)}{E_a} = 21.04\,ft$

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

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WALL B:

Story Shear due to Wind: $V_{4W} = 8548.56\,lb$ Story Shear due to Seismic: $F_2 = 9423.21\,lb$

Bldg Width in direction of Load: $L_t := 43.5\cdot ft$ Distance between shear walls: $L_1 := 23\cdot ft$

Shear Wall Length: $Lb_w := (7 + 2.75 + 3.25 + 6.5)ft = 19.5\,ft$ $Lb_s := (7 + 2.75 + 3.25 + 6.5)ft = 19.5\,ft$

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: $vb := \dfrac{vbb\cdot Lbb_w + \left(\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lb_w}$ Seismic Force: $\rho := 1.0$ $E_b := \dfrac{E_{bb}\cdot Lbb_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Lb_s}$

$vb = 148.91\,lb\cdot ft^{-1}$ $\frac{vb}{C_o} = 148.91\,lb\cdot ft^{-1}$ $E_b = 227.47\,lb\cdot ft^{-1}$ $\frac{E_b}{C_o} = 227.47\,lb\cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_b := 6.5\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_b := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1ft$ $DLRb := \frac{W_b\cdot L_b}{2}$ $DLRb = 325\,lb$

Chord Force:

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_w = 1340.18\,lb$ $CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_s = 2047.19\,lb$

$CFb_w + CFbb_w = 2040.82\,lb$ $CFb_s + CFbb_s = 3265.72\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 1145.18\,lb$ $HDFb_s := CFb_s - (0.6 - 0.14S_{DS})\cdot DLRb = 1896.84\,lb$

Simpson LSTHD8 or HDU2 w/ SSTB16 anchor

$HDFb_w + HDFbb_w = 1551.82\,lb$ $HDFb_s + HDFbb_s = 2888.69\,lb$

Simpson STHD10 or HDU4 w/ SSTB20 anchor

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 1.1\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.72\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vb} = 9.24\,ft$ $\frac{(Z_B\cdot C_o)}{E_b} = 6.05\,ft$

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

WALL C:

Story Shear due to Wind: $V_{2W} = 21931.37\,\text{lb}$ Story Shear due to Seismic: $F_2 = 9423.21\,\text{lb}$

Bldg Width in direction of Load: $L_t := 106\cdot\text{ft}$ Distance between shear walls: $L_1 := 17.5\cdot\text{ft}$ $L_2 := 13.5\text{ft}$

Shear Wall Length: $Lc_w := (11.25)\text{ft} = 11.25\,\text{ft}$ $Lc_s := (11.25)\text{ft} = 11.25\,\text{ft}$

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: $$vc := \frac{vcc\cdot Lcc_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lc_w}$$

Seismic Force: $\rho := 1.0$ $$E_c := \frac{E_{cc}\cdot Lcc_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lc_s}$$

$vc = 235.74\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{vc}{C_o} = 235.74\,\text{lb}\cdot\text{ft}^{-1}$ $E_c = 170.33\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{E_c}{C_o} = 170.33\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

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

$W_c := (15\cdot\text{psf})\cdot 1\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 9\text{ft}$ $DLRc := \frac{W_c\cdot L_c}{2}$ $DLRc = 1096.88\,\text{lb}$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_w = 2121.62\,\text{lb}$ $CFc_w + CFcc_w = 2650.97\,\text{lb}$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_s = 1532.94\,\text{lb}$ $CFc_s + CFcc_s = 2225.03\,\text{lb}$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 1463.5\,\text{lb}$
$HDFc_w + HDFcc_w = 1580.35\,\text{lb}$

$HDFc_s := CFc_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRc = 1025.51\,\text{lb}$
$HDFc_s + HDFcc_s = 1399.56\,\text{lb}$

Simpson HDU2 w/ PAB5 anchor

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

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

$B_n := \frac{\left(C_D\cdot Z_N\cdot C_o\right)}{vc} = 0.69\,\text{ft}$ $\frac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_c} = 0.96\,\text{ft}$

16d @ 8" o.c.

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

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

$As := \frac{\left(Z_B\cdot C_o\right)}{vc} = 5.84\,\text{ft}$ $\frac{\left(Z_B\cdot C_o\right)}{E_c} = 8.08\,\text{ft}$

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

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PROJECT : **Marbella Residence**

Phone: 253-858-3248
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WALL D:

Story Shear due to Wind: $V_{2W} = 21931.37\,lb$ Story Shear due to Seismic: $F_2 = 9423.21\,lb$

Bldg Width in direction of Load: $L_t := 106 \cdot ft$ Distance between shear walls: $L_1 := 32 \cdot ft$ $L_2 := 10ft$

Shear Wall Length: $Ld_w := (10.83 + 7.25 + 3.25)ft = 21.33\,ft$ $Ld_s := \left[10.83 + 7.25 + 3.25\left(\frac{6.5}{9}\right)\right]ft = 20.43\,ft$

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

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

Wind Force: $vd := \dfrac{vdd \cdot Ldd_w + \left(\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Ld_w}$

Seismic Force: $\rho := 1.0$ $E_d := \dfrac{E_{dd} \cdot Ldd_s + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Ld_s}$

$vd = 184.62\,lb \cdot ft^{-1}$ $\frac{vd}{C_o} = 184.62\,lb \cdot ft^{-1}$

$E_d = 149.16\,lb \cdot ft^{-1}$ $\frac{E_d}{C_o} = 149.16\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_d := 3.25 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 243.75\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 1661.55\,lb$

$CFd_w + CFdd_w = 2124.29\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 1342.44\,lb$

$CFd_s + CFdd_s = 1947.44\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 1515.3\,lb$

$HDFd_w + HDFdd_w = 1802.91\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 1229.68\,lb$

$HDFd_s + HDFdd_s = 1699.65\,lb$

SImpson LSTHD8 or HDU2 w/ SSTB16 or PAB5 anchor

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 0.88\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 1.09\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vd} = 7.45\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 9.22\,ft$

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

WALL E:

Story Shear due to Wind: $V_{2W} = 21931.37\,\text{lb}$ Story Shear due to Seismic: $F_2 = 9423.21\,\text{lb}$

Bldg Width in direction of Load: $L_t := 106\cdot\text{ft}$ Distance between shear walls: $L_1 := 32\cdot\text{ft}$ $L_2 := 17.5\text{ft}$

Shear Wall Length: $Le_w := (18)\text{ft} = 18\,\text{ft}$ $Le_s := (18)\text{ft} = 18\,\text{ft}$

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: $ve := \dfrac{vee\cdot Lee_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Le_w}$

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

$ve = 285.07\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{ve}{C_o} = 285.07\,\text{lb}\cdot\text{ft}^{-1}$ $E_e = 235.11\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{E_e}{C_o} = 235.11\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_e := 18\cdot\text{ft}$ Plate Height: $Pt := 9\cdot\text{ft}$

$W_e := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 17\text{ft}$

$DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 2340\,\text{lb}$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 2565.63\,\text{lb}$ $CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 2115.96\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 1161.63\,\text{lb}$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 1033.44\,\text{lb}$

$HDFe_w + HDFee_w = 1605.04\,\text{lb}$ $HDFe_s + HDFee_s = 1842.28\,\text{lb}$

Simpson LSTHD8RJ or HDU2 w/ PAB5 anchor (6" embed)

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.57\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.69\,\text{ft}$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{ve} = 4.83\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_e} = 5.85\,\text{ft}$

5/8" A.B. @ 54" o.c.

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WALL F:

Story Shear due to Wind: $V_{4W} = 8548.56\,\text{lb}$ Story Shear due to Seismic: $F_2 = 9423.21\,\text{lb}$

Bldg Width in direction of Load: $L_t := 43.5\cdot\text{ft}$ Distance between shear walls: $L_1 := 20.5\cdot\text{ft}$ $L_2 := 23\text{ft}$

Shear Wall Length:

$Lf_w := (8 + 11 + 15 + 6.5 + 2\cdot 3.125)\text{ft} = 46.75\,\text{ft}$

$Lf_s := \left[8 + 11 + 15 + 6.5 + 2\cdot 3.125\left(\frac{6.25}{9}\right)\right]\text{ft} = 44.84\,\text{ft}$

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: $vf := \dfrac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lf_w}$ Seismic Force: $\rho := 1.0$ $E_f := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lf_s}$

$vf = 54.86\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{vf}{C_o} = 54.86\,\text{lb}\cdot\text{ft}^{-1}$ $E_f = 73.55\,\text{lb}\cdot\text{ft}^{-1}$ $\frac{E_f}{C_o} = 73.55\,\text{lb}\cdot\text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_f := 3.125\cdot\text{ft}$ Plate Height: $Pt := 9\cdot\text{ft}$

$W_f := (15\cdot\text{psf})\cdot 4\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$ $DLRf := \frac{W_f\cdot L_f}{2}$ $DLRf = 234.37\,\text{lb}$

Chord Force:

$CFf_w := \frac{vf\cdot L_f\cdot Pt}{C_o\cdot L_f}$ $CFf_w = 493.71\,\text{lb}$ $CFf_s := \frac{E_f\cdot L_f\cdot Pt}{C_o\cdot L_f}$ $CFf_s = 661.97\,\text{lb}$

Holdown Force:

$HDFf_w := CFf_w - 0.6\cdot DLRf = 353.09\,\text{lb}$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS})\cdot DLRf = 553.55\,\text{lb}$

No Holdowns Required

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vf} = 2.98\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_f} = 2.22\,\text{ft}$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vf} = 25.08\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_f} = 18.71\,\text{ft}$

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

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WALL G:

Story Shear due to Wind: $V_{2W} = 21931.37\,\text{lb}$

Story Shear due to Seismic: $F_2 = 9423.21\,\text{lb}$

Bldg Width in direction of Load: $L_t := 106 \cdot \text{ft}$

Distance between shear walls: $L_1 := 13.5 \cdot \text{ft}$

Shear Wall Length: $Lg_w := (2 \cdot 2.583)\text{ft} = 5.17\,\text{ft}$

$$Lg_s := \left[2 \cdot 2.583 \left(\frac{5.167}{9}\right)\right]\text{ft} = 2.97\,\text{ft}$$

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: $$vg := \frac{\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1}{2}}{Lg_w}$$

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

$vg = 162.2\,\text{lb} \cdot \text{ft}^{-1}$ $\quad \frac{vg}{C_o} = 162.2\,\text{lb} \cdot \text{ft}^{-1}$

$E_g = 141.63\,\text{lb} \cdot \text{ft}^{-1}$ $\quad \frac{E_g}{C_o} = 141.63\,\text{lb} \cdot \text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_g := 2.583 \cdot \text{ft}$ Plate Height: $Pt := 9 \cdot \text{ft}$

$W_g := (15 \cdot \text{psf}) \cdot 3 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 0\text{ft}$

$DLRg := \frac{W_g \cdot L_g}{2}$ $\quad DLRg = 174.35\,\text{lb}$

Chord Force:

$CFg_w := \frac{vg \cdot L_g \cdot Pt}{C_o \cdot L_g}$ $\quad CFg_w = 1459.83\,\text{lb}$

$CFg_s := \frac{E_g \cdot L_g \cdot Pt}{C_o \cdot L_g}$ $\quad CFg_s = 1274.64\,\text{lb}$

Holdown Force:

$HDFg_w := CFg_w - 0.6 \cdot DLRg = 1355.22\,\text{lb}$

$HDFg_s := CFg_s - (0.6 - 0.14S_{DS}) \cdot DLRg = 1193.98\,\text{lb}$

Simpson LSTHD8RJ or HDU2 w/ SSTB16 anchor

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

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

$B_R := \frac{(C_D \cdot Z_N \cdot C_o)}{vg} = 1.01\,\text{ft} \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_g} = 1.15\,\text{ft}$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vg} = 8.48\,\text{ft} \quad \frac{(Z_B \cdot C_o)}{E_g} = 9.72\,\text{ft}$

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

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WALL H:

Story Shear due to Wind: $V_{2W} = 21931.37\,lb$ Story Shear due to Seismic: $F_2 = 9423.21\,lb$

Bldg Width in direction of Load: $L_t := 106 \cdot ft$ Distance between shear walls: $L_1 := 33 \cdot ft$ $L_2 := 10ft$

Shear Wall Length: $Lh_w := (12.92)ft = 12.92\,ft$ $Lh_s := (12.92)ft = 12.92\,ft$

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

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

Wind Force: $vh := \dfrac{\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lh_w}$ Seismic Force: $\rho := 1.0$ $E_h := \dfrac{\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lh_s}$

$vh = 206.58\,lb \cdot ft^{-1}$ $\frac{vh}{C_o} = 206.58\,lb \cdot ft^{-1}$ $E_h = 103.55\,lb \cdot ft^{-1}$ $\frac{E_h}{C_o} = 103.55\,lb \cdot ft^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_h := 12.92 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_h := (15 \cdot psf) \cdot 4 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRh := \frac{W_h \cdot L_h}{2}$ $DLRh = 969\,lb$

Chord Force:

$CFh_w := \frac{vh \cdot L_h \cdot Pt}{C_o \cdot L_h}$ $CFh_w = 1859.22\,lb$ $CFh_s := \frac{E_h \cdot L_h \cdot Pt}{C_o \cdot L_h}$ $CFh_s = 931.99\,lb$

Holdown Force:

$HDFh_w := CFh_w - 0.6 \cdot DLRh = 1277.82\,lb$ $HDFh_s := CFh_s - \left(0.6 - 0.14S_{DS}\right) \cdot DLRh = 483.71\,lb$

Simpson LSTHD8RJ or HDU2 w/ SSTB16 anchor

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

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

$B_R := \frac{\left(C_D \cdot Z_N \cdot C_o\right)}{vh} = 0.79\,ft$ $\frac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_h} = 1.58\,ft$

16d @ 8" o.c.

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

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

$As := \frac{\left(Z_B \cdot C_o\right)}{vh} = 6.66\,ft$ $\frac{\left(Z_B \cdot C_o\right)}{E_h} = 13.29\,ft$

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

WALL I:

Story Shear due to Wind: $V_{2W} = 21931.37\,\text{lb}$ Story Shear due to Seismic: $F_2 = 9423.21\,\text{lb}$

Bldg Width in direction of Load: $L_t := 106 \cdot \text{ft}$ Distance between shear walls: $L_1 := 33 \cdot \text{ft}$

Shear Wall Length: $Li_w := (4.83 + 7)\text{ft} = 11.83\,\text{ft}$ $Li_s := (4.83 + 7)\text{ft} = 11.83\,\text{ft}$

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

Wind Force: $vi := \dfrac{\dfrac{0.6V_{2W}}{L_t} \cdot \dfrac{L_1}{2}}{Li_w}$ Seismic Force: $\rho := 1.0$ $E_i := \dfrac{\rho \cdot \dfrac{0.7F_2}{L_t} \cdot \dfrac{L_1}{2}}{Li_s}$

$vi = 173.15\,\text{lb} \cdot \text{ft}^{-1}$ $\dfrac{vi}{C_o} = 173.15\,\text{lb} \cdot \text{ft}^{-1}$ $E_i = 86.79\,\text{lb} \cdot \text{ft}^{-1}$ $\dfrac{E_i}{C_o} = 86.79\,\text{lb} \cdot \text{ft}^{-1}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_i := 4.5 \cdot \text{ft}$ Plate Height: $Pt := 5 \cdot \text{ft}$

$W_i := (15 \cdot \text{psf}) \cdot 5 \cdot \text{ft} + (10 \cdot \text{psf}) \cdot Pt + (10\text{psf}) \cdot 0\text{ft}$ $DLRi := \dfrac{W_i \cdot L_i}{2}$ $DLRi = 281.25\,\text{lb}$

Chord Force:

$CFi_w := \dfrac{vi \cdot L_i \cdot Pt}{C_o \cdot L_i}$ $CFi_w = 865.73\,\text{lb}$ $CFi_s := \dfrac{E_i \cdot L_i \cdot Pt}{C_o \cdot L_i}$ $CFi_s = 433.97\,\text{lb}$

Holdown Force:

$HDFi_w := CFi_w - 0.6 \cdot DLRi = 696.98\,\text{lb}$ $HDFi_s := CFi_s - \left(0.6 - 0.14S_{DS}\right) \cdot DLRi = 303.86\,\text{lb}$

No Holdowns Required

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

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

$B_R := \dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{vi} = 0.94\,\text{ft}$ $\dfrac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_i} = 1.88\,\text{ft}$

16d @ 8" o.c.

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

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

$AS := \dfrac{\left(Z_B \cdot C_o\right)}{vi} = 7.95\,\text{ft}$ $\dfrac{\left(Z_B \cdot C_o\right)}{E_i} = 15.85\,\text{ft}$

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

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$vaa \cdot \frac{Laa_w}{49.5ft} = 31.27 lb \cdot ft^{-1}$ $\quad E_{aa} \cdot \frac{Laa_s}{49.5ft} = 54.38 lb \cdot ft^{-1}$

Wall Lines BB:

$vbb \cdot \frac{Lbb_w}{32ft} = 48.37 lb \cdot ft^{-1}$ $\quad E_{bb} \cdot \frac{Lbb_s}{32ft} = 84.12 lb \cdot ft^{-1}$

Wall Lines CC:

$vcc \cdot \frac{Lcc_w}{11ft} = 66.17 lb \cdot ft^{-1}$ $\quad E_{cc} \cdot \frac{Lcc_s}{11ft} = 86.51 lb \cdot ft^{-1}$

Wall Lines DD:

$vdd \cdot \frac{Ldd_w}{34ft} = 39.15 lb \cdot ft^{-1}$ $\quad E_{dd} \cdot \frac{Ldd_s}{34ft} = 51.18 lb \cdot ft^{-1}$

Wall Lines EE:

$vee \cdot \frac{Lee_w}{41.5ft} = 49.61 lb \cdot ft^{-1}$ $\quad E_{ee} \cdot \frac{Lee_s}{41.5ft} = 64.86 lb \cdot ft^{-1}$

Wall Lines A:

$\frac{va \cdot La_w - vaa \cdot Laa_w}{106ft} = 11.4 lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La_s - E_{aa} \cdot Laa_s}{106ft} = 14.66 lb \cdot ft^{-1}$ $\quad \frac{va \cdot La_w}{106ft} = 26 lb \cdot ft^{-1}$ $\quad \frac{E_a \cdot La_s}{106ft} = 40.06 lb \cdot ft^{-1}$

Wall Lines B:

$\frac{vb \cdot Lb_w - vbb \cdot Lbb_w}{32ft} = 42.37 lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb_s - E_{bb} \cdot Lbb_s}{32ft} = 54.49 lb \cdot ft^{-1}$ $\quad \frac{vb \cdot Lb_w}{32ft} = 90.74 lb \cdot ft^{-1}$ $\quad \frac{E_b \cdot Lb_s}{32ft} = 138.61 lb \cdot ft^{-1}$

Wall Lines C:

$\frac{vc \cdot Lc_w - vcc \cdot Lcc_w}{36ft} = 53.45 lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc_s - E_{cc} \cdot Lcc_s}{36ft} = 26.79 lb \cdot ft^{-1}$ $\quad \frac{vc \cdot Lc_w}{36ft} = 73.67 lb \cdot ft^{-1}$ $\quad \frac{E_c \cdot Lc_s}{36ft} = 53.23 lb \cdot ft^{-1}$

Wall Lines D:

$\frac{vd \cdot Ld_w - vdd \cdot Ldd_w}{26.5ft} = 98.37 lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld_s - E_{dd} \cdot Ldd_s}{26.5ft} = 49.31 lb \cdot ft^{-1}$ $\quad \frac{vd \cdot Ld_w}{26.5ft} = 148.6 lb \cdot ft^{-1}$ $\quad \frac{E_d \cdot Ld_s}{26.5ft} = 114.98 lb \cdot ft^{-1}$

Wall Line E:

$\frac{ve \cdot Le_w - vee \cdot Lee_w}{41ft} = 74.94 lb \cdot ft^{-1}$ $\quad \frac{E_e \cdot Le_s - E_{ee} \cdot Lee_s}{41ft} = 37.56 lb \cdot ft^{-1}$ $\quad \frac{ve \cdot Le_w}{41ft} = 125.15 lb \cdot ft^{-1}$ $\quad \frac{E_e \cdot Le_s}{41ft} = 103.22 lb \cdot ft^{-1}$

Wall Line F:

$\frac{vf \cdot Lf_w}{106ft} = 24.19 lb \cdot ft^{-1}$ $\quad \frac{E_f \cdot Lf_s}{106ft} = 31.11 lb \cdot ft^{-1}$

Wall Line G:

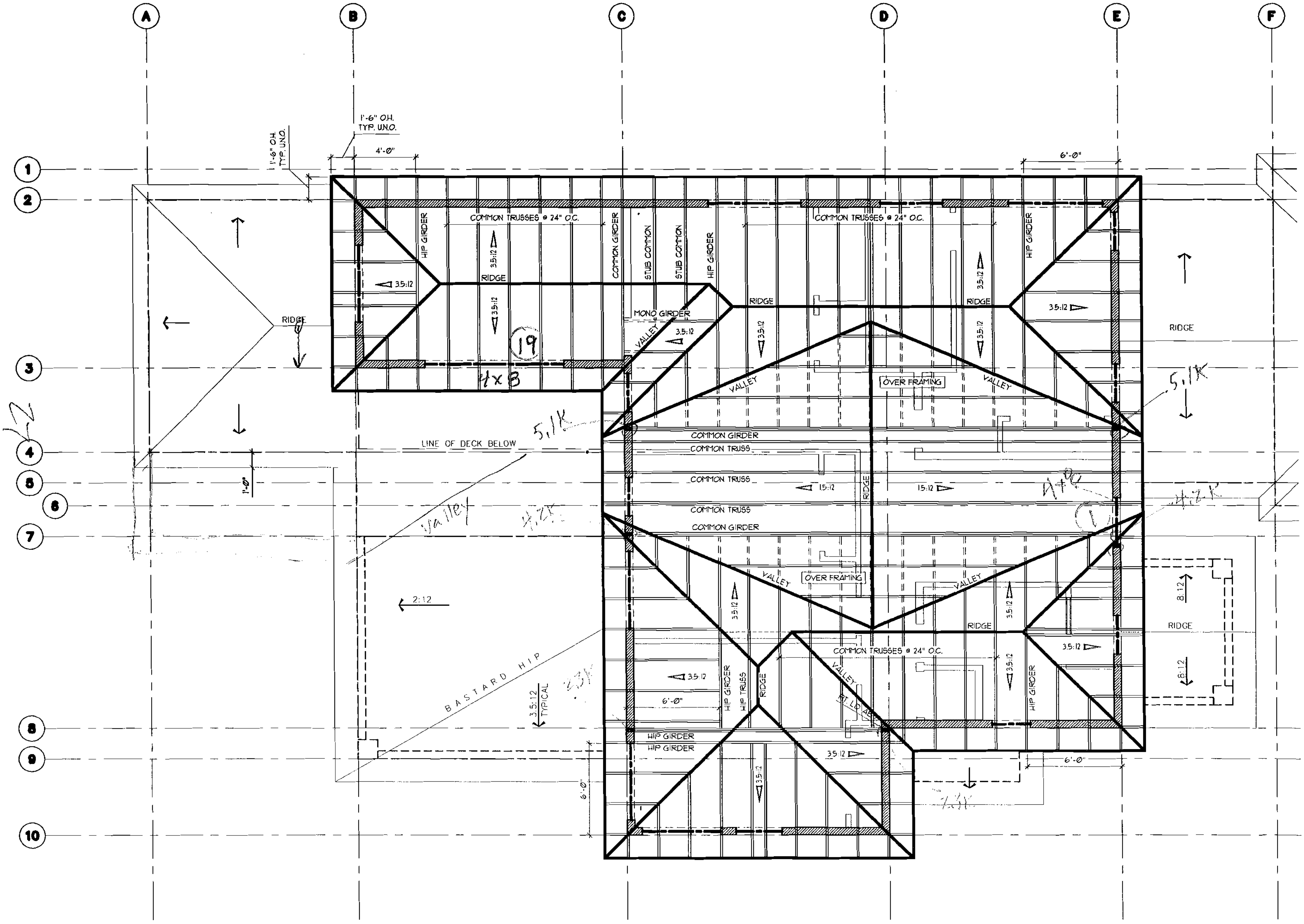
$\frac{vg \cdot Lg_w}{22ft} = 38.09 lb \cdot ft^{-1}$ $\quad \frac{E_g \cdot Lg_s}{22ft} = 19.09 lb \cdot ft^{-1}$

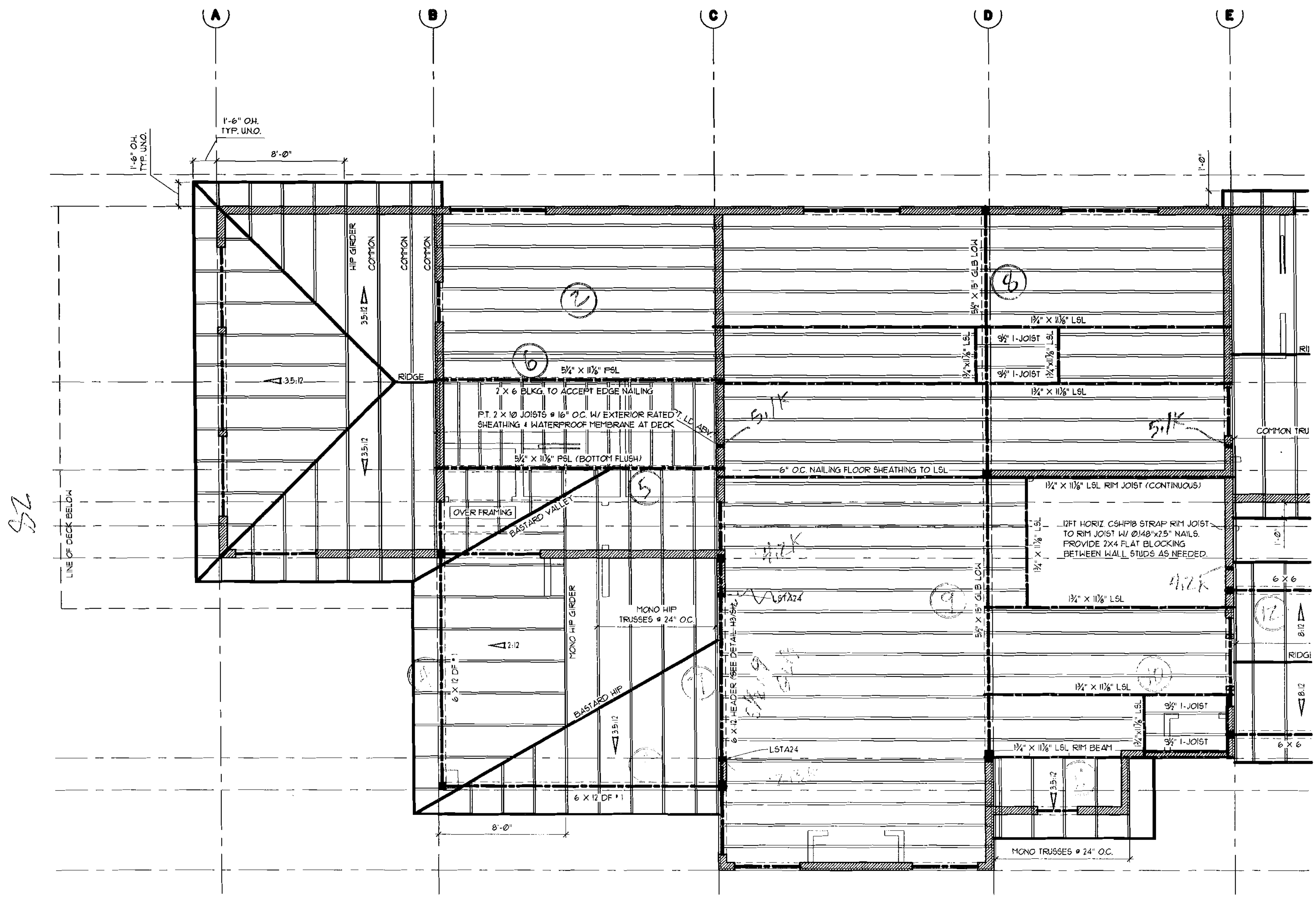
Wall Line H:

$\frac{vh \cdot Lh_w}{22ft} = 121.32 lb \cdot ft^{-1}$ $\quad \frac{E_h \cdot Lh_s}{22ft} = 60.81 lb \cdot ft^{-1}$

Wall Line I:

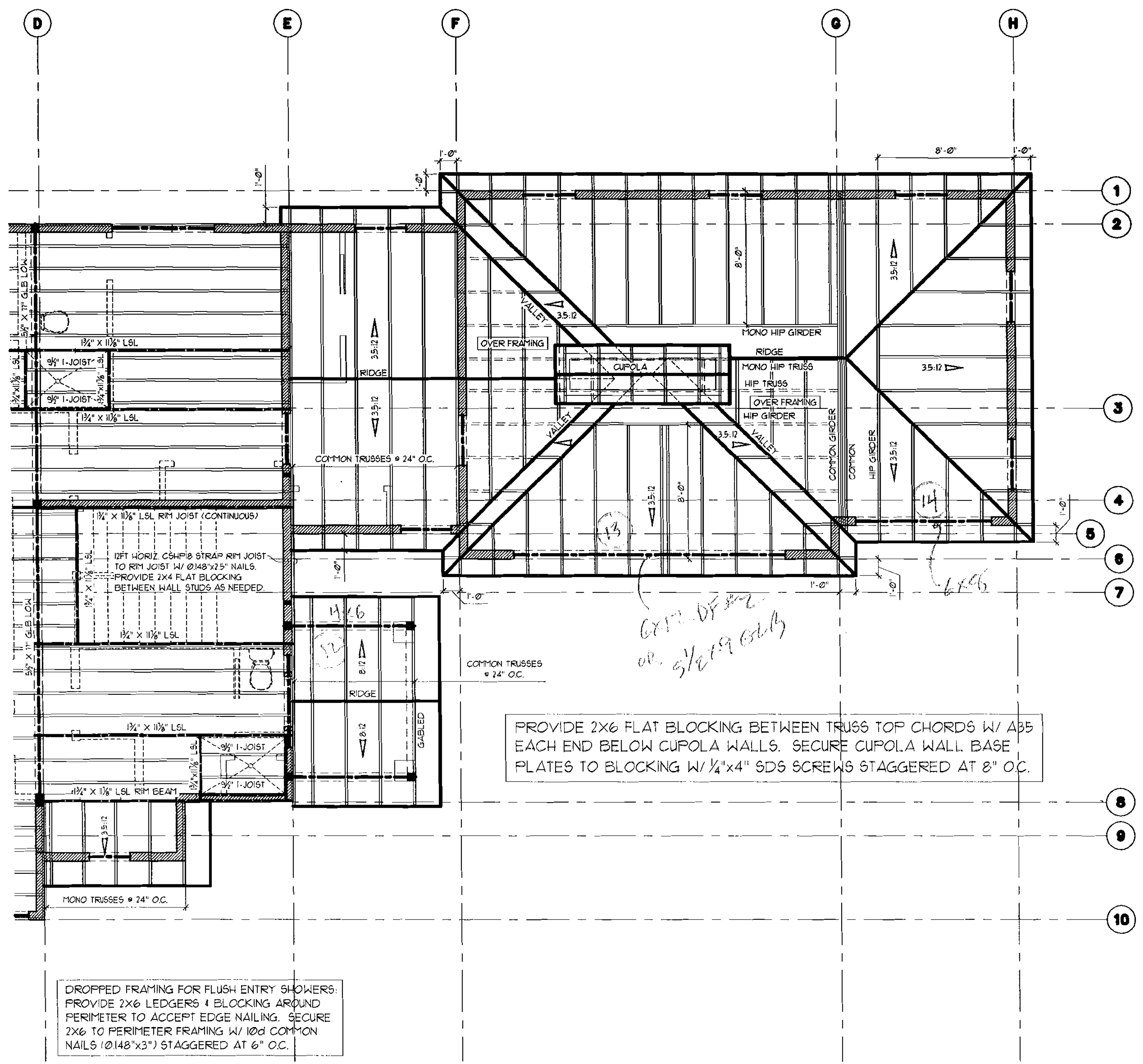
$\frac{vi \cdot Li_w}{20ft} = 102.42 lb \cdot ft^{-1}$ $\quad \frac{E_i \cdot Li_s}{20ft} = 51.34 lb \cdot ft^{-1}$


A
B
C
D
E
F
1
2
3
4
5
6
7
8
9
10
1'-6" O.H. TYP. U.N.O.
4'-0"
6'-0"
COMMON TRUSSES @ 24" O.C.
HIP GIRDER
COMMON GIRDER
STUB COMMON
MONO GIRDER
RIDGE
VALLEY
3.5:12
OVER FRAMING
COMMON TRUSS
15:12
LINE OF DECK BELOW
1'-0"
2:12
BASTARD HIP
3.5:12 TYPICAL
HIP TRUSS
8:12
RT. LD. ABV.
19
4x8
5,1K
4.2K
2.3K
Valley

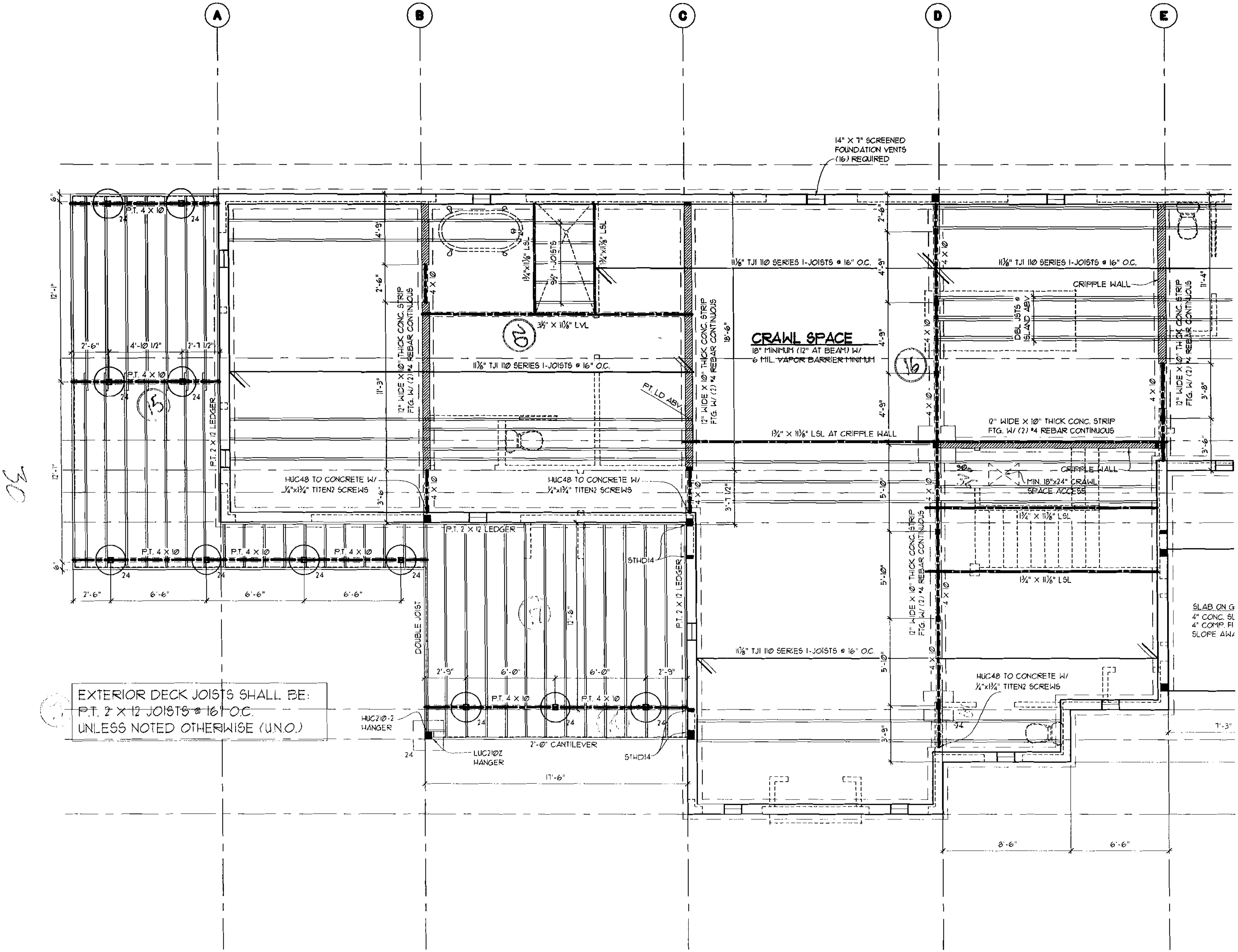


UPPER FLOOR JOISTS SHALL BE:
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)

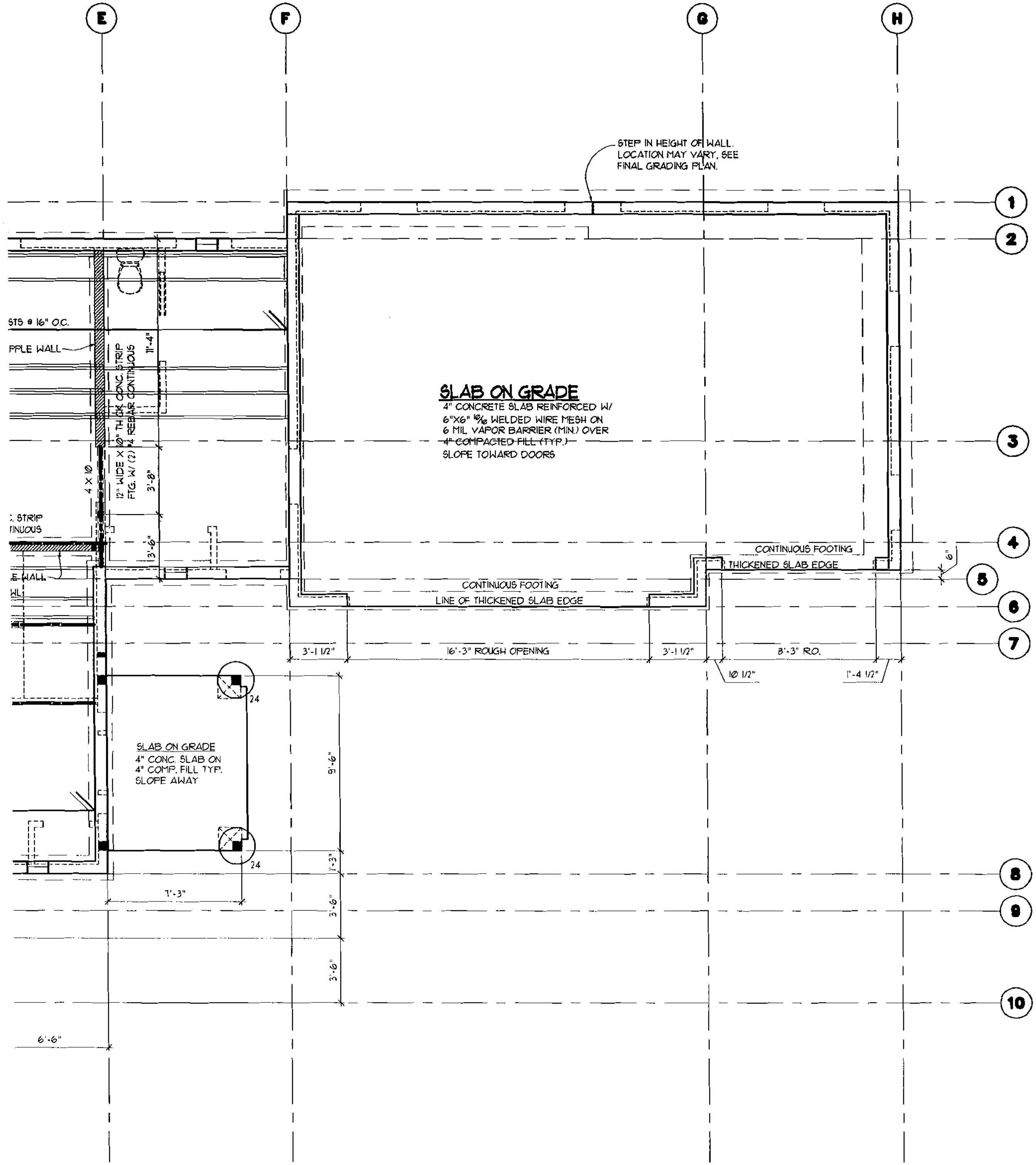
DROPPED FRAMING FOR FLUSH ENTRY SHOWERS:
PROVIDE 2X6 LEDGERS & BLOCKING AROUND
PERIMETER TO ACCEPT EDGE NAILING. SECURE
2X6 TO PERIMETER FRAMING W/ 10d COMMON
NAILS (Ø.148"x3") STAGGERED AT 6" O.C.



PROVIDE 2X6 FLAT BLOCKING BETWEEN TRUSS TOP CHORDS W/ A35 EACH END BELOW CUPOLA WALLS. SECURE CUPOLA WALL BASE PLATES TO BLOCKING W/ 1/4"x4" SDS SCREWS STAGGERED AT 8" O.C.
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.
12FT HORIZ. CSHP18 STRAP RIM JOIST TO RIM JOIST W/ 0.148"x2.5" NAILS. PROVIDE 2X4 FLAT BLOCKING BETWEEN WALL STUDS AS NEEDED.
1 3/4" X 11 7/8" LSL RIM JOIST (CONTINUOUS)
1 3/4" X 11 7/8" LSL RIM BEAM
1 3/4" X 11 7/8" LSL
9 1/2" I-JOIST
5 1/2" X 11" GLB LOW
COMMON TRUSSES @ 24" O.C.
MONO TRUSSES @ 24" O.C.
MONO HIP GIRDER
MONO HIP TRUSS
HIP TRUSS
HIP GIRDER
COMMON GIRDER
COMMON
RIDGE
VALLEY
CUPOLA
OVER FRAMING
GABLED
3.5:12
8:12
8'-0"
1'-0"
D
E
F
G
H
1
2
3
4
5
6
7
8
9
10
6x6
4x6


A
B
C
D
E
14" X 7" SCREENED FOUNDATION VENTS (16) REQUIRED
CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/ 6 MIL. VAPOR BARRIER MINIMUM
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
12" WIDE X 10" THICK CONC. STRIP FTG. W/ (2) #4 REBAR CONTINUOUS
1¾" X 11⅞" LSL AT CRIPPLE WALL
CRIPPLE WALL
DBL JSTS @ ISLAND ABV
MIN. 18"x24" CRAWL SPACE ACCESS
1¾" X 11⅞" LSL
3½" X 11⅞" LVL
9½" I-JOISTS
1¾"x11⅞" LSL
P.T. LD ABV
HUC48 TO CONCRETE W/ ¼"x1¾" TITEN2 SCREWS
P.T. 2 X 12 LEDGER
P.T. 4 X 10
DOUBLE JOIST
STHD14
HUC210-2 HANGER
LUC210Z HANGER
2'-0" CANTILEVER
SLAB ON G
4" CONC. SL
4" COMP. FI
SLOPE AW
EXTERIOR DECK JOISTS SHALL BE:
P.T. 2 X 12 JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
30

31


E
F
G
H
1
2
3
4
5
6
7
8
9
10
STEP IN HEIGHT OF WALL. LOCATION MAY VARY, SEE FINAL GRADING PLAN.
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED FILL (TYP.)
SLOPE TOWARD DOORS
CONTINUOUS FOOTING
THICKENED SLAB EDGE
CONTINUOUS FOOTING
LINE OF THICKENED SLAB EDGE
3'-1 1/2"
16'-3" ROUGH OPENING
3'-1 1/2"
8'-3" R.O.
10 1/2"
1'-4 1/2"
6"
12" WIDE X 10" THICK CONC. STRIP FTG. W/ (2) #4 REBAR CONTINUOUS
11'-4"
3'-8"
3'-6"
4 X 10
STS @ 16" O.C.
PPLE WALL
. STRIP
TINUOUS
E WALL
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
24
24
9'-6"
1'-3"
3'-6"
3'-6"
7'-3"
6'-6"

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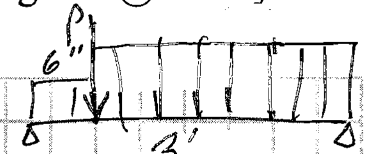
① $w_{D_1} = 15psf\,(35'/2) = 262.5plf$
$w_{S_1} = 25psf\,(35'/2) = 437.5plf$

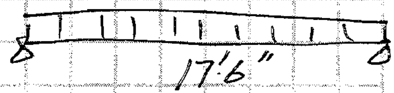
$P = 1700^{\#}DL + 2500^{\#}SL$ from Girder

$w_{D2} = 15psf\,(9'/2) = 67.5plf$
$w_{S2} = 25psf\,(9'/2) = 112.5plf$



4x8
(4x8 w/ w_1 only & 6' span)

② $w_D = 15psf$
$w_L = 40psf$

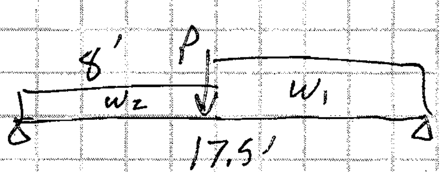


11 7/8 TJI 110 @16" o.c.

③ $w_{D_1} = 15psf\,(18'/2) = 135plf$
$w_{S_1} = 25psf\,(18'/2) = 225plf$

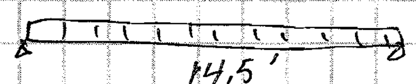
$P = 530^{\#}DL + 775^{\#}SL$ from hipgirder

$w_{D2} = 15psf\,(4'/2) = 30plf$
$w_{S2} = 25psf\,(4'/2) = 50plf$



6x12 DF#1

④ $w_D = 15psf\,(11'/2) = 82.5plf$
$w_S = 25psf\,(11'/2) = 137.5plf$



6x10 minimum

⑤ $w_D = 15psf\,(11'/2) = 82.5plf$
$w_L = 60psf\,(5.5'/2) = 165plf$
$w_S = 25psf\,(11'/2) = 137.5plf$

5 1/2x12 GLB
or
5 1/4x11 7/8 LVL

FOR Marbella
JOB

DATE 8-12-20
BY MM

MYERS ENGINEERING

Myers Engineering LLC
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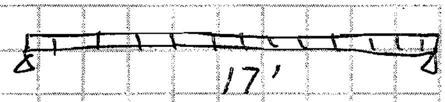
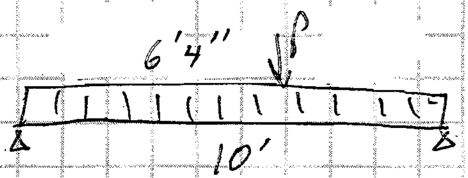
Gig Harbor, WA 98335
(253) 858-3248
myengineer@centurytel.net

(6) $w_D = 15psf(14'/2 + 5.5'/2 + 1') + 12psf(9') = 269.25plf$
$w_L = 60psf(5.5'/2) + 40psf(1') = 205plf$
$w_S = 25psf(14'/2 + 5.5'/2) = 243.75plf$



5¼x11⅞ PSL

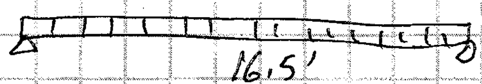
(7) $w_D = 15psf(17'/2 + 1' + 9'/2) + 12psf(9') = 318plf$
$w_L = 40psf(17'/2) = 340plf$
$w_S = 25psf(1' + 9'/2) = 137.5plf$



5½x 9" GLB

$P = \pm 875^{\#} WL \pm 1140^{\#} EL$ w/ $\Omega = 3.0$

(8) $w_D = 15psf(31'/2) = 232.5plf$
$w_L = 40psf(31'/2) = 620plf$

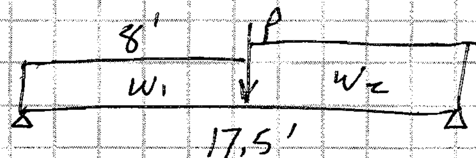


5½x15 GLB

(9) $w_{D1} = 15psf(19'/2) = 142.5plf$
$w_{L1} = 40psf(19'/2) = 380plf$

$w_{D2} = 15psf(31'/2) = 232.5plf$
$w_{L2} = 40psf(31'/2) = 620plf$

$P = 290^{\#} DL + 760^{\#} LL$ from stair rim



5½x15 GLB

FOR Marbella
JOB

DATE 8-12-20
BY [initials]

MYERS ENGINEERING

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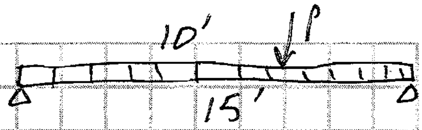
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
myengineer@centurytel.net

(10) $w_D = 15\,plf$
$w_L = 40\,plf$

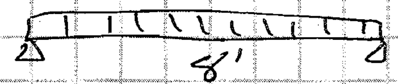
$P = 225^{\#}DL + 600^{\#}LL$



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

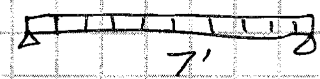
(11) $w_D = 15\,psf\left(\frac{15.5'}{2} + \frac{3.5'}{2} + 1'\right) + 12\,psf\,(9') = 265.5\,plf$
$w_L = 40\,plf$
$w_S = 25\,psf\left(\frac{15.5'}{2} + \frac{3.5'}{2}\right) = 237.5\,plf$

$P = \pm 710^{\#}WL \pm 1220^{\#}EL$ w/ $\Omega = 3.0$



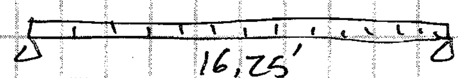
$1\frac{3}{4} \times 11\frac{7}{8}$ LSL

(12) $w_D = 15\,psf\left(\frac{13'}{2}\right) = 97.5\,plf$
$w_S = 25\,psf\left(\frac{13'}{2}\right) = 162.5\,plf$



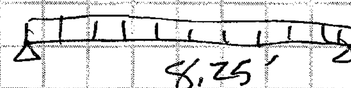
4x6 Minimum

(13) $w_D = 15\,psf\left(\frac{11'}{2}\right) = 82.5\,plf$
$w_S = 25\,psf\left(\frac{11'}{2}\right) = 137.5\,plf$



~~6x12~~
$5\frac{1}{2} \times 9$ GLB

(14) $w_D = 15\,psf\left(\frac{22'}{2}\right) = 165\,plf$
$w_S = 25\,psf\left(\frac{22'}{2}\right) = 275\,plf$



6x8

FOR Marbella
JOB

DATE 8-12-10
BY [signature]

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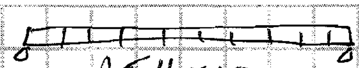
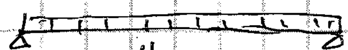
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
myengineer@centurytel.net

(15) $w_D = 10psf\,(24'/2) = 120plf$
$w_L = 60psf\,(24'/2) = 720plf$
$w_S = 25psf\,(24'/2) = 300plf$



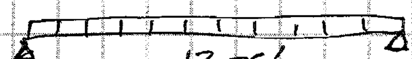
P.T. 4x10
5'8" Max span

(16) $w_D = 15psf\,(31'/2) = 232.5plf$
$w_L = 40psf\,(31'/2) = 620plf$



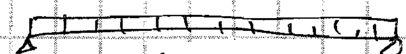
4x10
6'5" clear span Max

(17) $w_D = 10psf$
$w_L = 60psf$



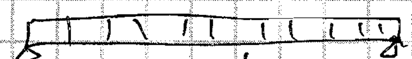
12.25'
P.T. 2x12 @ 16" o.c.
(2x6 @ 6' span)

(18) $w_D = 10psf\,(16.5'/2) = 82.5plf$
$w_L = 60psf\,(16.5'/2) = 495plf$



P.T. 4x10
6'1" clear span Max

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

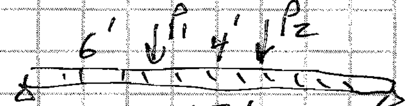


9'
4x8

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

$P_1 = 300^{\#}DL + 800^{\#}LL$

$P_2 = 330^{\#}DL + 880^{\#}LL$



17'
3 1/2 x 11 7/8 LVL

FOR Marbella
JOB

DATE 8-13-20
BY MM

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

Wood Beam

File: Marbella.ec6
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DESCRIPTION: 1. Upper Header

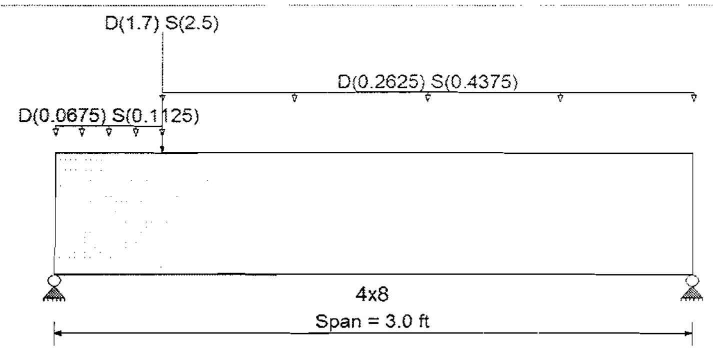
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	900 psi	Ebend- xx	1600 ksi
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	180 psi		
		Ft	575 psi	Density	31.21 pcf

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.06750, S = 0.1125 k/ft, Extent = 0.0 -->> 0.50 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.2625, S = 0.4375 k/ft, Extent = 0.50 -->> 3.0 ft, Tributary Width = 1.0 ft
Point Load : D = 1.70, S = 2.50 k @ 0.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.621**: 1	Maximum Shear Stress Ratio	=	**0.373** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	835.05 psi		=	77.25 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	0.526 ft	Location of maximum on span	=	2.398 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.011 in	Ratio =	3273 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.018 in	Ratio =	1984 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.312	1.728
Overall MINimum	2.591	1.059
D Only	1.721	0.669
+D+L	1.721	0.669
+D+S	4.312	1.728
+D+0.750L	1.721	0.669
+D+0.750L+0.750S	3.664	1.463
+0.60D	1.033	0.401
S Only	2.591	1.059

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

Wood Beam

File: Marbella.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

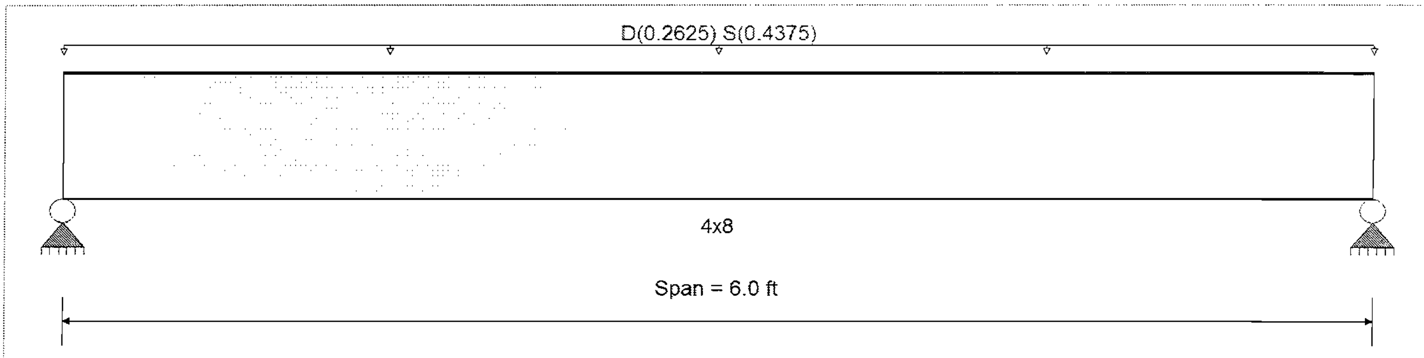
DESCRIPTION: 1a. Upper Header

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.2625, S = 0.4375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.916: 1**	Maximum Shear Stress Ratio	=	**0.482 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,232.82psi		=	99.67 psi
	=	1,345.50psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.072 in	Ratio =	997 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.115 in	Ratio =	623 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.100	2.100
Overall MINimum	1.313	1.313
D Only	0.788	0.788
+D+L	0.788	0.788
+D+S	2.100	2.100
+D+0.750L	0.788	0.788
+D+0.750L+0.750S	1.772	1.772
+0.60D	0.473	0.473
S Only	1.313	1.313

L/480 Live Load Deflection

2

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

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

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

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

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

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

How to Use These Tables

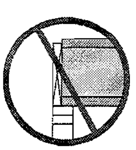
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

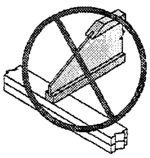
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor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

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

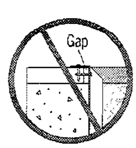
These Conditions Are <u>NOT</u> Permitted:



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



DO NOT bevel cut joist beyond inside face of wall.



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

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Wood Beam

DESCRIPTION: 3. Porch Roof Beam (South)

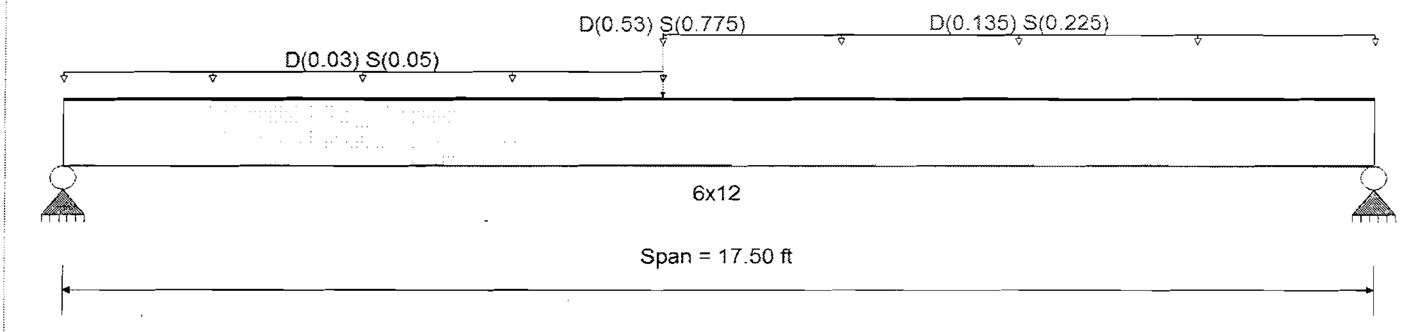
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.030, S = 0.050 k/ft, Extent = 0.0 -->> 8.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.530, S = 0.7750 k @ 8.0 ft
Uniform Load : D = 0.1350, S = 0.2250 k/ft, Extent = 8.0 -->> 17.50 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.926 1**	Maximum Shear Stress Ratio	=	**0.351 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	1,438.38 psi		=	68.53 psi
	=	1,552.50 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.495 ft	Location of maximum on span	=	16.542 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.418 in	Ratio =	502 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.680 in	Ratio =	308 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.130	3.235
Overall MINimum	1.309	2.003
D Only	0.821	1.232
+D+L	0.821	1.232
+D+S	2.130	3.235
+D+0.750L	0.821	1.232
+D+0.750L+0.750S	1.803	2.734
+0.60D	0.493	0.739
S Only	1.309	2.003

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Wood Beam

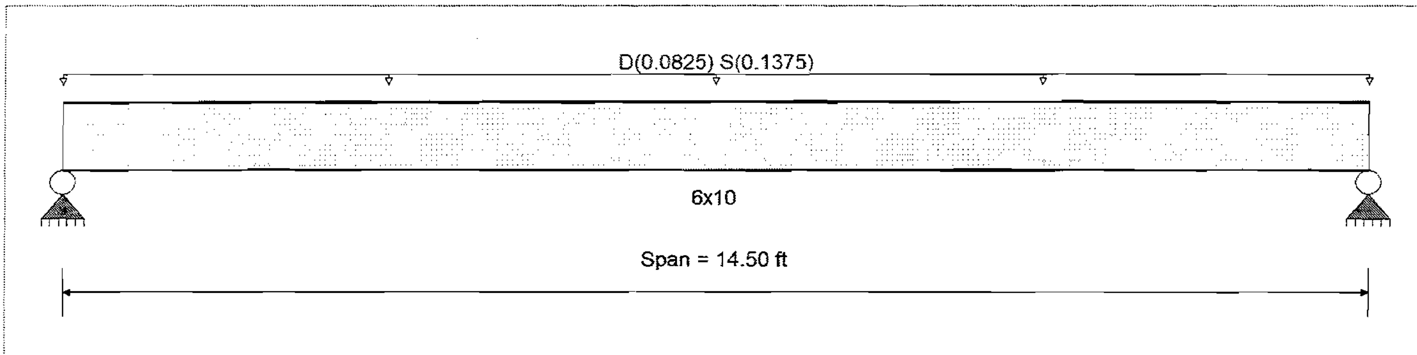
DESCRIPTION: 4. Porch Roof Beam (West)

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.08250, S = 0.1375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.833 : 1**	Maximum Shear Stress Ratio	=	**0.210 : 1**
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	838.67 psi		=	41.11 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.269 in	Ratio =	646 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.431 in	Ratio =	403 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.595	1.595
Overall MINimum	0.997	0.997
D Only	0.598	0.598
+D+L	0.598	0.598
+D+S	1.595	1.595
+D+0.750L	0.598	0.598
+D+0.750L+0.750S	1.346	1.346
+0.60D	0.359	0.359
S Only	0.997	0.997

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Wood Beam

DESCRIPTION: 5. Deck beam at Grid 4

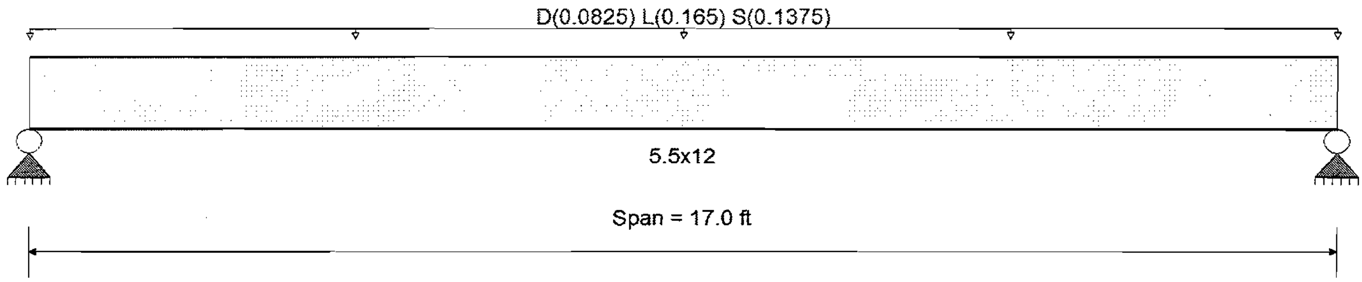
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.08250, L = 0.1650, S = 0.1375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.368: 1**	Maximum Shear Stress Ratio	=	**0.173 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,016.02 psi		=	52.79 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.219 in	Ratio =	932 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.410 in	Ratio =	497 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.630	2.630
Overall MINimum	1.169	1.169
D Only	0.701	0.701
+D+L	2.104	2.104
+D+S	1.870	1.870
+D+0.750L	1.753	1.753
+D+0.750L+0.750S	2.630	2.630
+0.60D	0.421	0.421
L Only	1.403	1.403
S Only	1.169	1.169

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Wood Beam

DESCRIPTION: 5. Deck beam at Grid 4

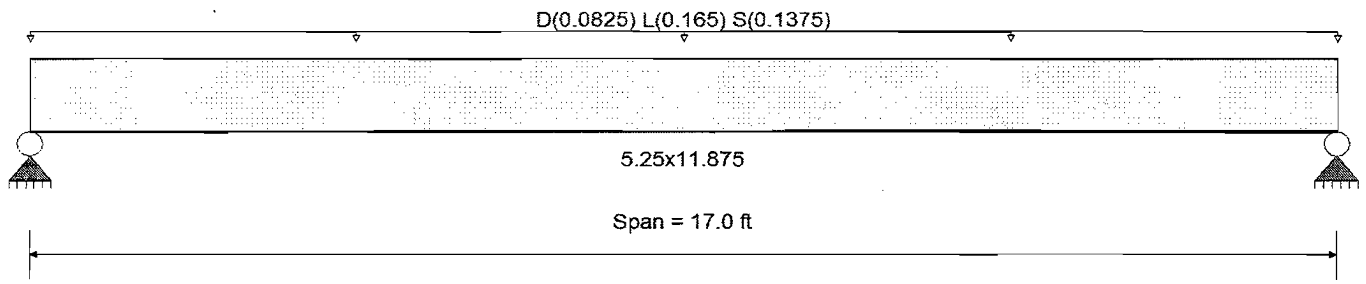
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2600 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2600 psi	Ebend- xx	1900 ksi
		Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285 psi		
		Ft	1555 psi	Density	42.01 pcf

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



Applied Loads

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

Uniform Load : D = 0.08250, L = 0.1650, S = 0.1375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.364** 1	Maximum Shear Stress Ratio	=	**0.172** : 1
Section used for this span		**5.25x11.875**	Section used for this span		**5.25x11.875**
	=	1,086.92 psi		=	56.34 psi
	=	2,990.00 psi		=	327.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.224 in	Ratio =	910 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.420 in	Ratio =	485 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.630	2.630
Overall MINimum	1.169	1.169
D Only	0.701	0.701
+D+L	2.104	2.104
+D+S	1.870	1.870
+D+0.750L	1.753	1.753
+D+0.750L+0.750S	2.630	2.630
+0.60D	0.421	0.421
L Only	1.403	1.403
S Only	1.169	1.169

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Wood Beam

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DESCRIPTION: 6. Beam at Grid 3

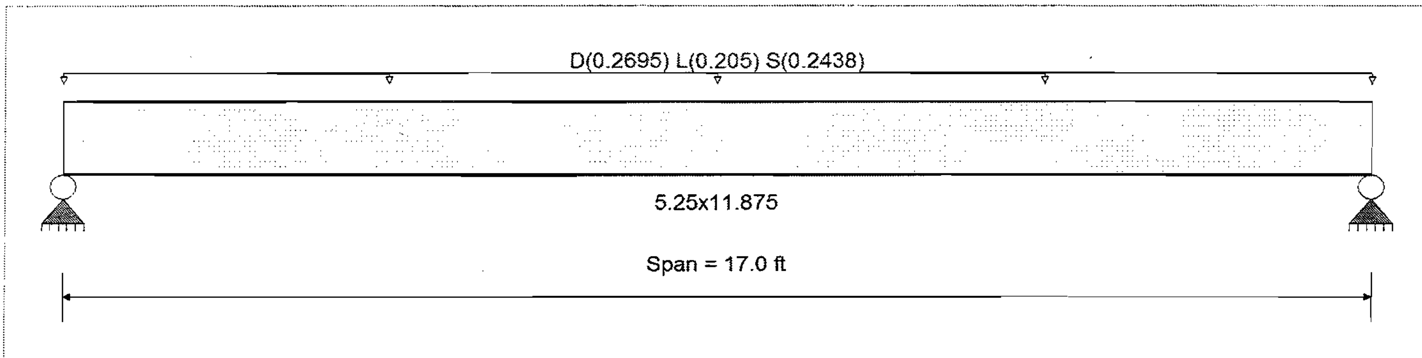
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2900 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - xx	1016.535 ksi
Wood Species	: Trus Joist	Fc - Perp	625 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 pcf

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



Applied Loads

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

Uniform Load : D = 0.2695, L = 0.2050, S = 0.2438 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.639 : 1**	Maximum Shear Stress Ratio	=	**0.331 : 1**
Section used for this span		**5.25x11.875**	Section used for this span		**5.25x11.875**
	=	2,129.40 psi		=	110.38 psi
	=	3,335.00 psi		=	333.50 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.315 in	Ratio =	648 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.782 in	Ratio =	260 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.152	5.152
Overall MINimum	2.072	2.072
D Only	2.291	2.291
+D+L	4.033	4.033
+D+S	4.363	4.363
+D+0.750L	3.598	3.598
+D+0.750L+0.750S	5.152	5.152
+0.60D	1.374	1.374
L Only	1.743	1.743
S Only	2.072	2.072

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Wood Beam

DESCRIPTION: 7. Header at Great Room (Grid C)

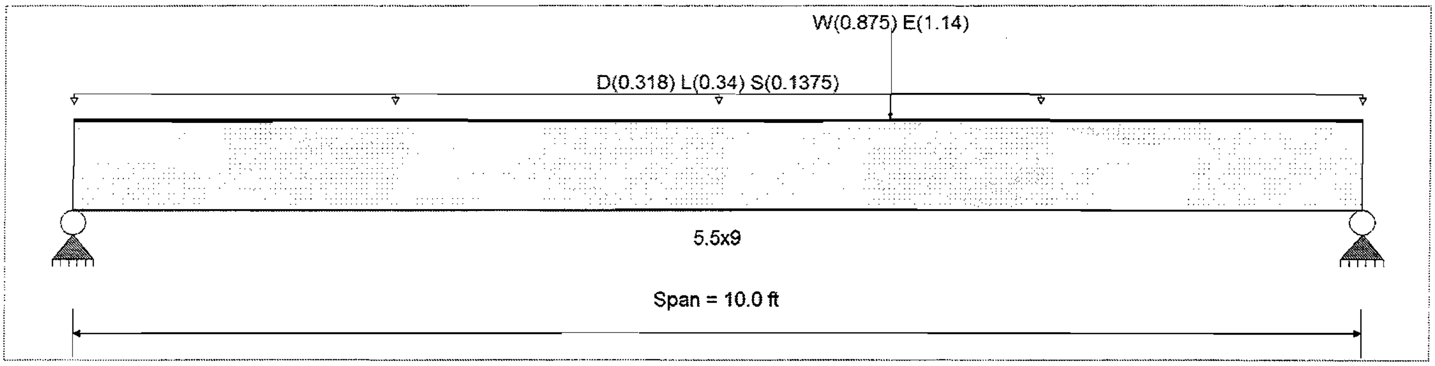
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.3180, L = 0.340, S = 0.1375 , Tributary Width = 1.0 ft
Point Load : W = 0.8750, E = 1.140 k @ 6.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.554** : 1	Maximum Shear Stress Ratio	=	**0.321** : 1
Section used for this span		**5.5x9**	Section used for this span		**5.5x9**
	=	1,329.29 psi		=	85.14 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.128 in	Ratio =	937 >=480
Max Upward Transient Deflection	-0.062 in	Ratio =	1924 >=480
Max Downward Total Deflection	0.287 in	Ratio =	418 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.600	3.760
Overall MINimum	-0.418	-0.722
D Only	1.590	1.590
+D+L	3.290	3.290
+D+S	2.278	2.278
+D+0.750L	2.865	2.865
+D+0.750L+0.750S	3.381	3.381
+D+0.60W	1.783	1.922
+D-0.60W	1.397	1.258
+D+0.70E	1.883	2.095
+D-0.70E	1.297	1.085
+D+0.750L+0.450W	3.009	3.114
+D+0.750L-0.450W	2.721	2.616
+D+0.750L+0.750S+0.450W	3.525	3.630

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Wood Beam

DESCRIPTION: 7. Header at Great Room (Grid C)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S-0.450W	3.236	3.131
+D+0.750L+0.750S+0.5250E	3.600	3.760
+D+0.750L+0.750S-0.5250E	3.161	3.002
+0.60D+0.60W	1.147	1.286
+0.60D-0.60W	0.761	0.622
+0.60D+0.70E	1.247	1.459
+0.60D-0.70E	0.661	0.449
L Only	1.700	1.700
S Only	0.688	0.688
W Only	0.321	0.554
-W	-0.321	-0.554
E Only	0.418	0.722
E Only * -1.0	-0.418	-0.722

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Wood Beam

DESCRIPTION: 8. Beam over Kitchen at Grid D

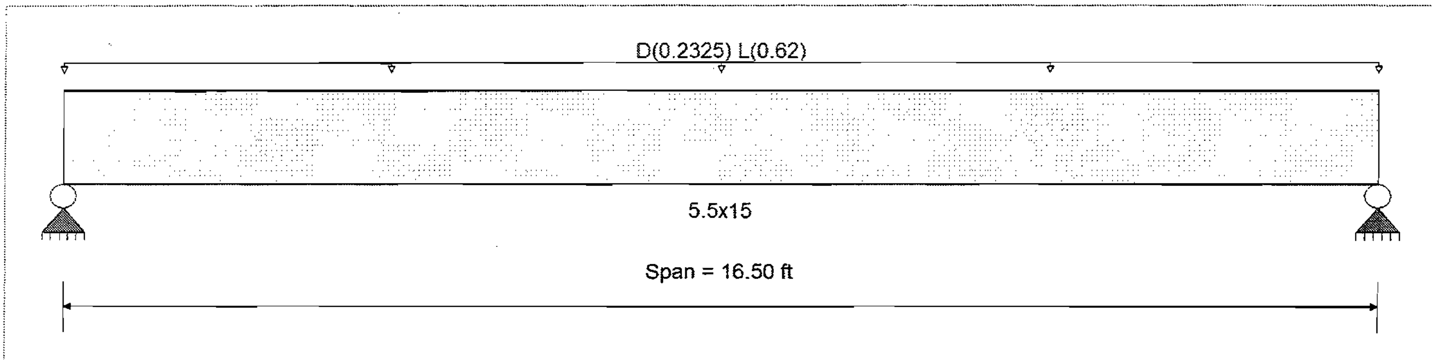
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.2325, L = 0.620 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.707**: 1	Maximum Shear Stress Ratio	=	**0.412** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	1,687.95 psi		=	109.21 psi
	=	2,387.41 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.374 in	Ratio =	530 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.514 in	Ratio =	385 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	7.033	7.033
Overall MINimum	5.115	5.115
D Only	1.918	1.918
+D+L	7.033	7.033
+D+S	1.918	1.918
+D+0.750L	5.754	5.754
+D+0.750L+0.750S	5.754	5.754
+0.60D	1.151	1.151
L Only	5.115	5.115
S Only		

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DESCRIPTION: 9. Beam over Great Rm at Grid D

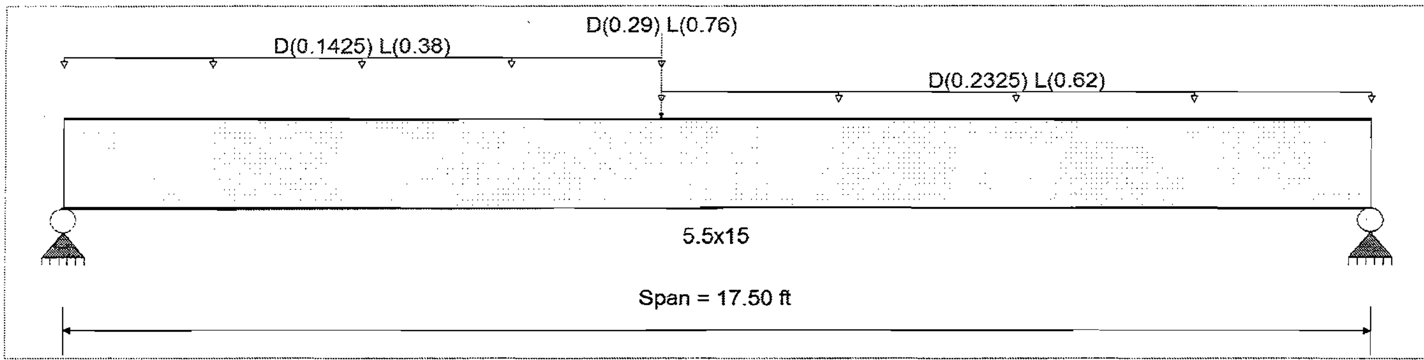
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.2325, L = 0.620 k/ft, Extent = 8.0 -->> 17.50 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.1425, L = 0.380 k/ft, Extent = 0.0 -->> 8.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.290, L = 0.760 k @ 8.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.774** : 1	Maximum Shear Stress Ratio	=	**0.432** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	1,836.43 psi		=	114.57 psi
	=	2,373.40 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.878 ft	Location of maximum on span	=	16.286 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.446 in	Ratio =	470 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.614 in	Ratio =	342 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.993	7.336
Overall MINimum	4.356	5.334
D Only	1.636	2.002
+D+L	5.993	7.336
+D+S	1.636	2.002
+D+0.750L	4.904	6.003
+D+0.750L+0.750S	4.904	6.003
+0.60D	0.982	1.201
L Only	4.356	5.334
S Only		

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Wood Beam

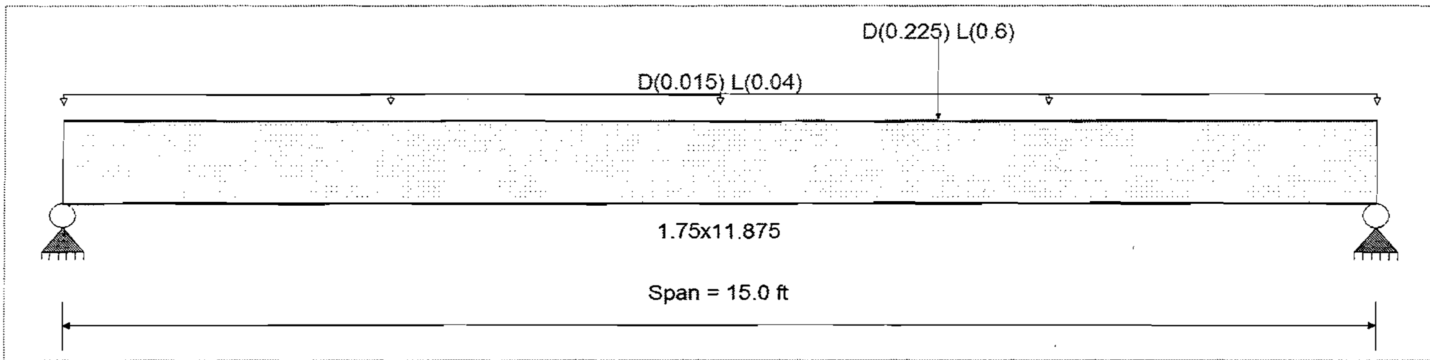
DESCRIPTION: 10. floor beam at shower

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2325 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2325 psi	Ebend- xx	1550 ksi
		Fc - Prll	2170 psi	Eminbend - xx	787.815 ksi
Wood Species	: Trus Joist	Fc - Perp	900 psi		
Wood Grade	: TimberStrand LSL 1.55E	Fv	310 psi		
		Ft	1070 psi	Density	45.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.2250, L = 0.60 k @ 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.517**: 1	Maximum Shear Stress Ratio	=	**0.211** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,202.04 psi		=	65.56 psi
	=	2,325.00 psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.964 ft	Location of maximum on span	=	14.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.287 in	Ratio =	627 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.394 in	Ratio =	456 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	0.688	0.963
Overall MINimum	0.500	0.700
D Only	0.188	0.263
+D+L	0.688	0.963
+D+S	0.188	0.263
+D+0.750L	0.563	0.788
+D+0.750L+0.750S	0.563	0.788
+0.60D	0.113	0.158
L Only	0.500	0.700
S Only		

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Wood Beam

DESCRIPTION: 11. Rim beam at Grid 8

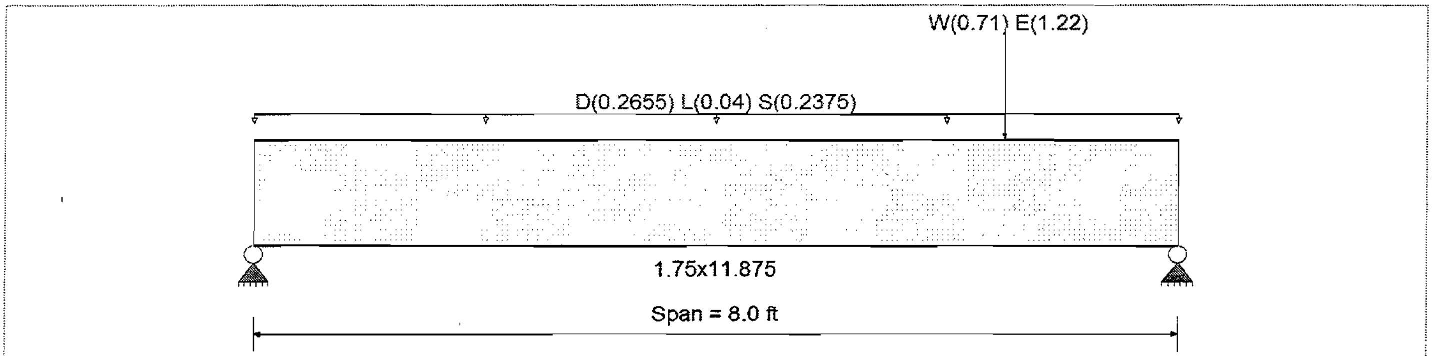
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.2655, L = 0.040, S = 0.2375 , Tributary Width = 1.0 ft
Point Load : W = 0.710, E = 1.220 k @ 6.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.439 : 1	Maximum Shear Stress Ratio	=	**0.449** : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
	=	1,174.05 psi		=	222.61 psi
	=	2,673.75 psi		=	496.00 psi
Load Combination		+D+S	Load Combination		+1.105D+0.750L+0.750S+1.575E
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.058 in	Ratio =	1650 >=480
Max Upward Transient Deflection	-0.033 in	Ratio =	2939 >=480
Max Downward Total Deflection	0.133 in	Ratio =	722 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.015	2.415
Overall MINimum	-0.229	-0.991
D Only	1.062	1.062
+D+L	1.222	1.222
+D+S	2.012	2.012
+D+0.750L	1.182	1.182
+D+0.750L+0.750S	1.895	1.895
+D+0.60W	1.142	1.408
+D-0.60W	0.982	0.716
+D+0.70E	1.222	1.756
+D-0.70E	0.902	0.368
+D+0.750L+0.450W	1.242	1.442
+D+0.750L-0.450W	1.122	0.922
+D+0.750L+0.750S+0.450W	1.954	2.154

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DESCRIPTION: 11. Rim beam at Grid 8

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S-0.450W	1.835	1.635
+D+0.750L+0.750S+0.5250E	2.015	2.415
+D+0.750L+0.750S-0.5250E	1.774	1.374
+0.60D+0.60W	0.717	0.983
+0.60D-0.60W	0.557	0.291
+0.60D+0.70E	0.797	1.331
+0.60D-0.70E	0.477	-0.057
L Only	0.160	0.160
S Only	0.950	0.950
W Only	0.133	0.577
-W	-0.133	-0.577
E Only	0.229	0.991
E Only * -1.0	-0.229	-0.991

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Wood Beam

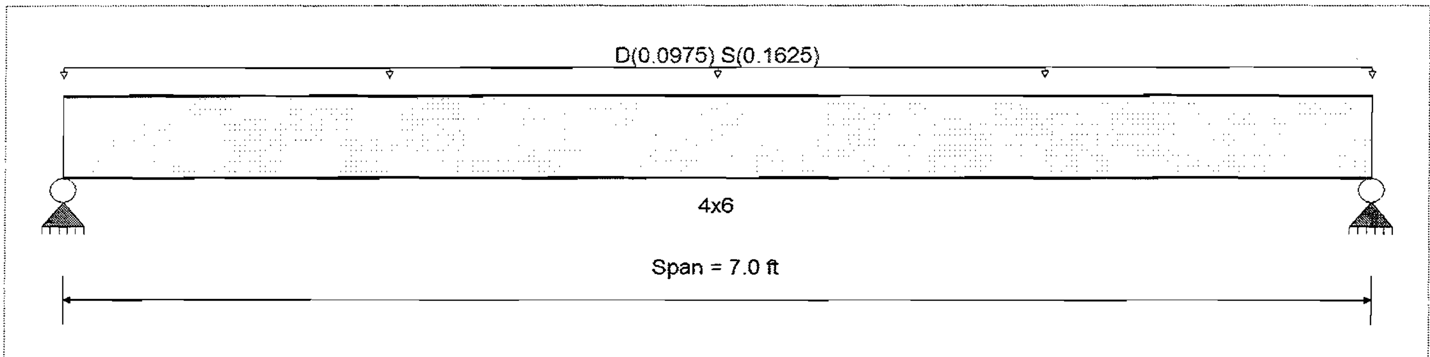
DESCRIPTION: 12. Entry Roof beam

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900 psi	Ebend- xx	1600 ksi
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.2	Fv	180 psi		
		Ft	575 psi	Density	31.21 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 6.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.805**: 1	Maximum Shear Stress Ratio	=	**0.300** : 1
Section used for this span		**4x6**	Section used for this span		**4x6**
	=	1,082.98 psi		=	62.11 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.566 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.114 in	Ratio =	738 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.182 in	Ratio =	461 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	0.910	0.910
Overall MINimum	0.569	0.569
D Only	0.341	0.341
+D+L	0.341	0.341
+D+S	0.910	0.910
+D+0.750L	0.341	0.341
+D+0.750L+0.750S	0.768	0.768
+0.60D	0.205	0.205
S Only	0.569	0.569

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Wood Beam

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DESCRIPTION: 13. 2 Car Door header

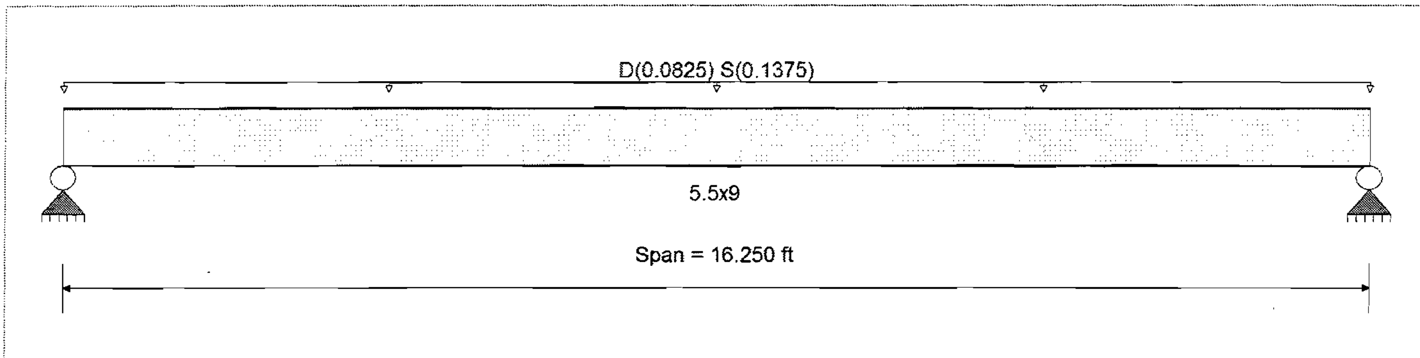
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.08250, S = 0.1375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.425: 1**	Maximum Shear Stress Ratio	=	**0.162 : 1**
Section used for this span		**5.5x9**	Section used for this span		**5.5x9**
	=	1,173.61 psi		=	49.42 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.361 in	Ratio =	540 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.577 in	Ratio =	337 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.788	1.788
Overall MINimum	1.117	1.117
D Only	0.670	0.670
+D+L	0.670	0.670
+D+S	1.788	1.788
+D+0.750L	0.670	0.670
+D+0.750L+0.750S	1.508	1.508
+0.60D	0.402	0.402
S Only	1.117	1.117

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Wood Beam

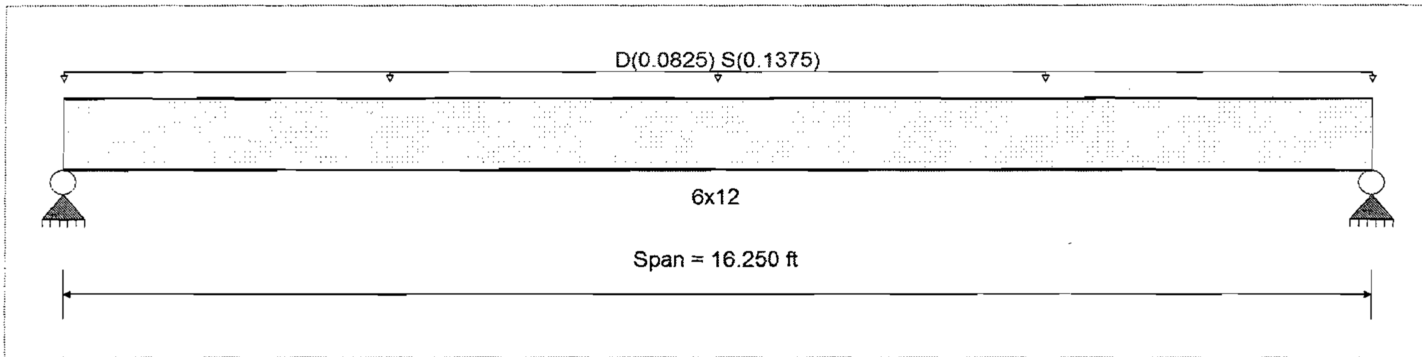
DESCRIPTION: 13. 2 Car Door header

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.08250, S = 0.1375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.714**: 1	Maximum Shear Stress Ratio	=	**0.192** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	718.81 psi		=	37.44 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.239 in	Ratio =	814 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.383 in	Ratio =	508 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.788	1.788
Overall MINimum	1.117	1.117
D Only	0.670	0.670
+D+L	0.670	0.670
+D+S	1.788	1.788
+D+0.750L	0.670	0.670
+D+0.750L+0.750S	1.508	1.508
+0.60D	0.402	0.402
S Only	1.117	1.117

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Wood Beam

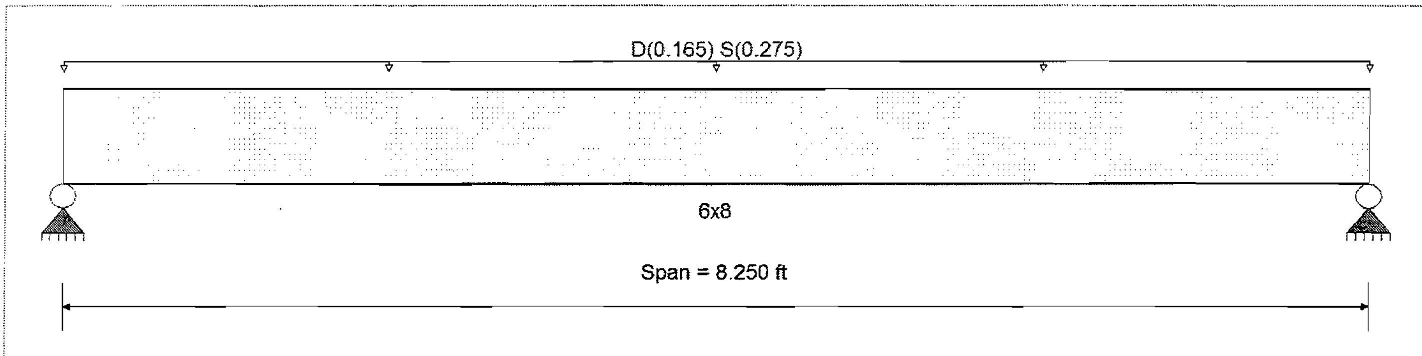
DESCRIPTION: 14. 3rd Car Header

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.1650, S = 0.2750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.866** : 1	Maximum Shear Stress Ratio	=	**0.288** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
	=	871.20 psi		=	56.36 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.115 in	Ratio =	863 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.184 in	Ratio =	539 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.815	1.815
Overall MINimum	1.134	1.134
D Only	0.681	0.681
+D+L	0.681	0.681
+D+S	1.815	1.815
+D+0.750L	0.681	0.681
+D+0.750L+0.750S	1.531	1.531
+0.60D	0.408	0.408
S Only	1.134	1.134

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Wood Beam

File: Marbella.ec6
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DESCRIPTION: 15. Deck beam at Master

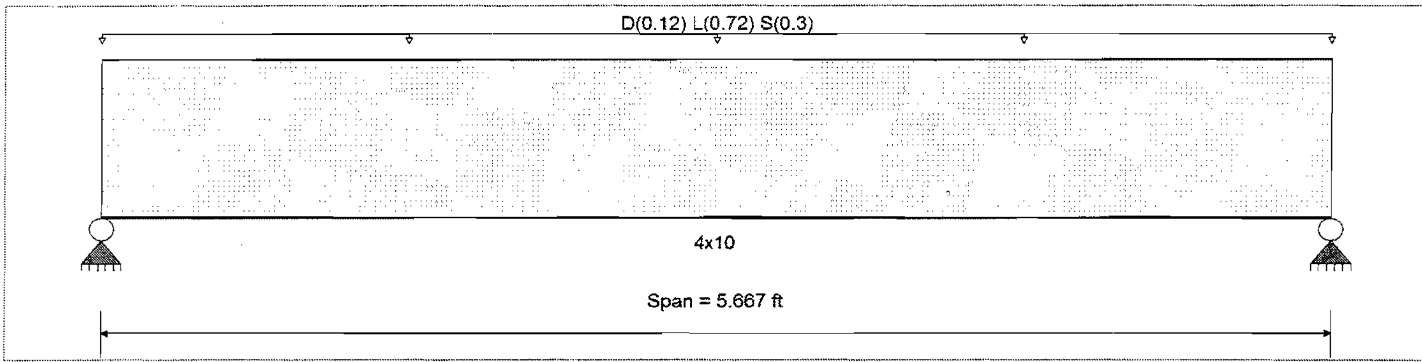
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850 psi	Ebend- xx	1300 ksi
		Fc - Prll	1300 psi	Eminbend - xx	470 ksi
Wood Species	: Hem Fir	Fc - Perp	405 psi		
Wood Grade	: No.2	Fv	150 psi		
		Ft	525 psi	Density	26.84 pcf

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



Applied Loads

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

Uniform Load : D = 0.120, L = 0.720, S = 0.30 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.994**: 1	Maximum Shear Stress Ratio	=	**0.671** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	810.73psi		=	80.49 psi
	=	816.00psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.834ft	Location of maximum on span	=	4.902 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.059 in	Ratio =	1153 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.072 in	Ratio =	938 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.508	2.508
Overall MINimum	0.850	0.850
D Only	0.340	0.340
+D+L	2.380	2.380
+D+S	1.190	1.190
+D+0.750L	1.870	1.870
+D+0.750L+0.750S	2.508	2.508
+0.60D	0.204	0.204
L Only	2.040	2.040
S Only	0.850	0.850

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Wood Beam

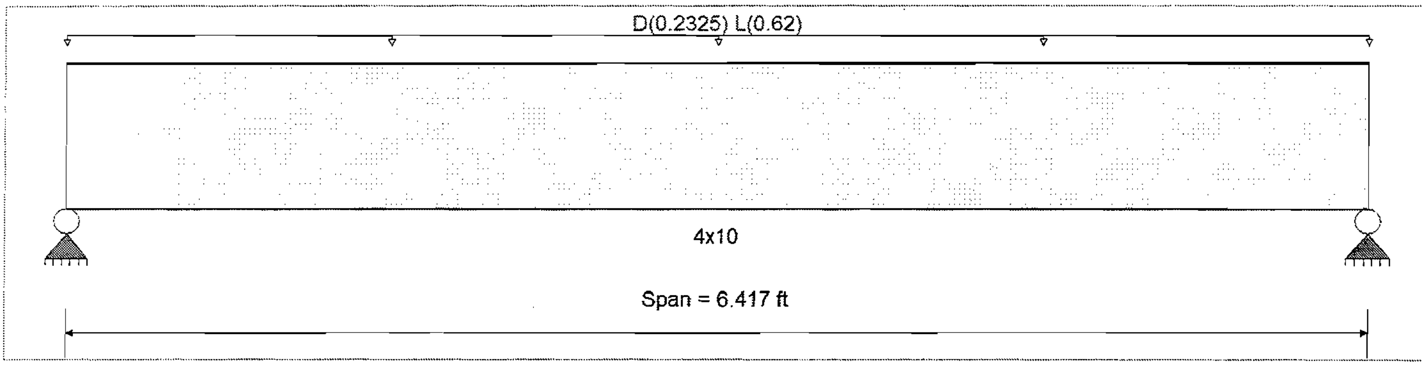
DESCRIPTION: 16. Crawl beam NOT at bearing wall

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: DouglasFir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.20 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.2325, L = 0.620 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.977**: 1	Maximum Shear Stress Ratio	=	**0.540** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,054.99 psi		=	97.13 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.209 ft	Location of maximum on span	=	5.668 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.064 in	Ratio =	1195 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.089 in	Ratio =	869 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	2.735	2.735
Overall MINimum	1.989	1.989
D Only	0.746	0.746
+D+L	2.735	2.735
+D+S	0.746	0.746
+D+0.750L	2.238	2.238
+D+0.750L+0.750S	2.238	2.238
+0.60D	0.448	0.448
L Only	1.989	1.989
S Only		

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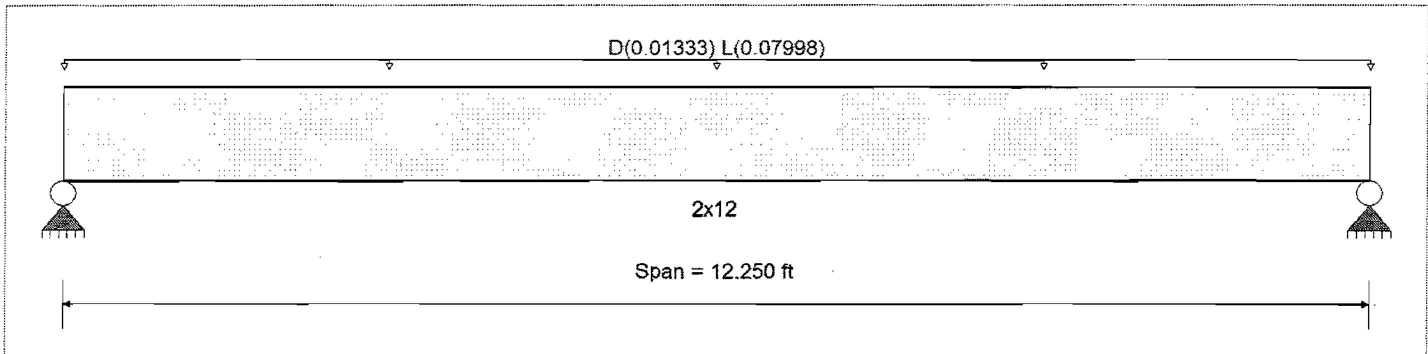
DESCRIPTION: 17. Deck Joist

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.849: 1**	Maximum Shear Stress Ratio	=	**0.362 : 1**
Section used for this span		**2x12**	Section used for this span		**2x12**
	=	663.81 psi		=	43.39 psi
	=	782.00 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.125 ft	Location of maximum on span	=	11.356 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.185 in	Ratio =	792 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.216 in	Ratio =	679 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.572	0.572
Overall MINimum	0.490	0.490
D Only	0.082	0.082
+D+L	0.572	0.572
+D+S	0.082	0.082
+D+0.750L	0.449	0.449
+D+0.750L+0.750S	0.449	0.449
+0.60D	0.049	0.049
L Only	0.490	0.490
S Only		

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MYERS ENGINEERING

Wood Beam

Lic. # : KW-06008232 MYERS ENGINEERING

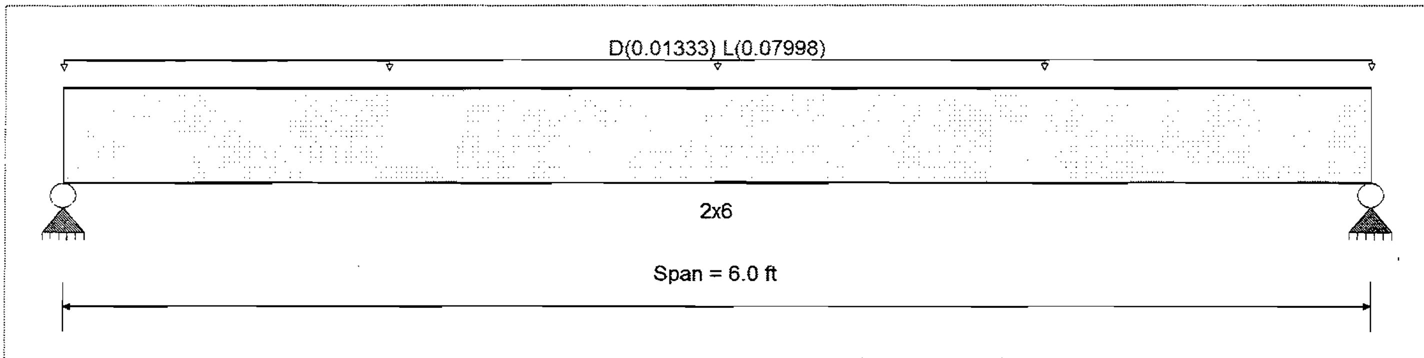
DESCRIPTION: 17a. Deck Joist

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.655 : 1**	Maximum Shear Stress Ratio	=	**0.362 : 1**
Section used for this span		**2x6**	Section used for this span		**2x6**
	=	666.28 psi		=	43.47 psi
	=	1,016.60 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.091 in	Ratio =	788 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.107 in	Ratio =	675 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.280	0.280
Overall MINimum	0.240	0.240
D Only	0.040	0.040
+D+L	0.280	0.280
+D+S	0.040	0.040
+D+0.750L	0.220	0.220
+D+0.750L+0.750S	0.220	0.220
+0.60D	0.024	0.024
L Only	0.240	0.240
S Only		

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Wood Beam

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DESCRIPTION: 18. Rear Porch Deck Beam

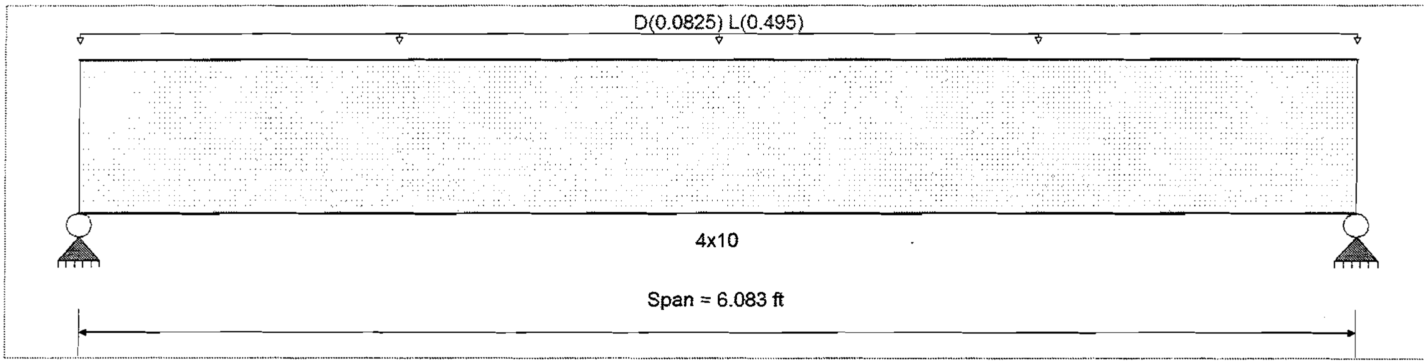
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	675 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	675 psi	Ebend- xx	1100 ksi
		Fc - Prll	500 psi	Eminbend - xx	400 ksi
Wood Species	: Hem Fir	Fc - Perp	405 psi		
Wood Grade	: No.2	Fv	140 psi		
		Ft	350 psi	Density	26.84 pcf

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



Applied Loads

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

Uniform Load : D = 0.08250, L = 0.4950 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.991**: 1	Maximum Shear Stress Ratio	=	**0.546** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	642.21 psi		=	61.18 psi
	=	648.00 psi		=	112.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.042ft	Location of maximum on span	=	5.328 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.064 in	Ratio =	1147 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.074 in	Ratio =	983 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.756	1.756
Overall MINimum	1.506	1.506
D Only	0.251	0.251
+D+L	1.756	1.756
+D+S	0.251	0.251
+D+0.750L	1.380	1.380
+D+0.750L+0.750S	1.380	1.380
+0.60D	0.151	0.151
L Only	1.506	1.506
S Only		

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 18a. Rear Porch Deck Beam

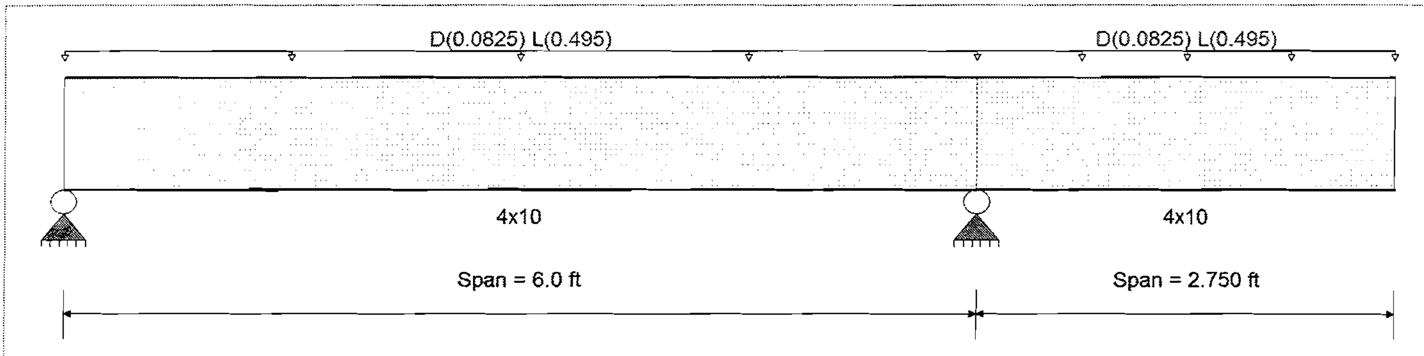
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	675.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	675.0 psi	Ebend- xx	1,100.0 ksi
		Fc - Prll	500.0 psi	Eminbend - xx	400.0 ksi
Wood Species	: Hem Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	140.0 psi		
		Ft	350.0 psi	Density	26.840 pcf

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.08250, L = 0.4950 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.08250, L = 0.4950 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.810** : 1	Maximum Shear Stress Ratio	=	**0.691** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	525.01 psi		=	77.40 psi
	=	648.00 psi		=	112.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.000 ft	Location of maximum on span	=	5.263 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.031 in	Ratio =	2355 >=480
Max Upward Transient Deflection	-0.001 in	Ratio =	59050 >=480
Max Downward Total Deflection	0.036 in	Ratio =	2018 >=360
Max Upward Total Deflection	-0.001 in	Ratio =	50614 >=360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.369	3.685	
Overall MINimum	1.173	3.158	
D Only	0.196	0.526	
+D+L	1.369	3.685	
+D+S	0.196	0.526	
+D+0.750L	1.075	2.895	
+D+0.750L+0.750S	1.075	2.895	
+0.60D	0.117	0.316	
L Only	1.173	3.158	
S Only			

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Wood Beam

DESCRIPTION: 19. Upper Header

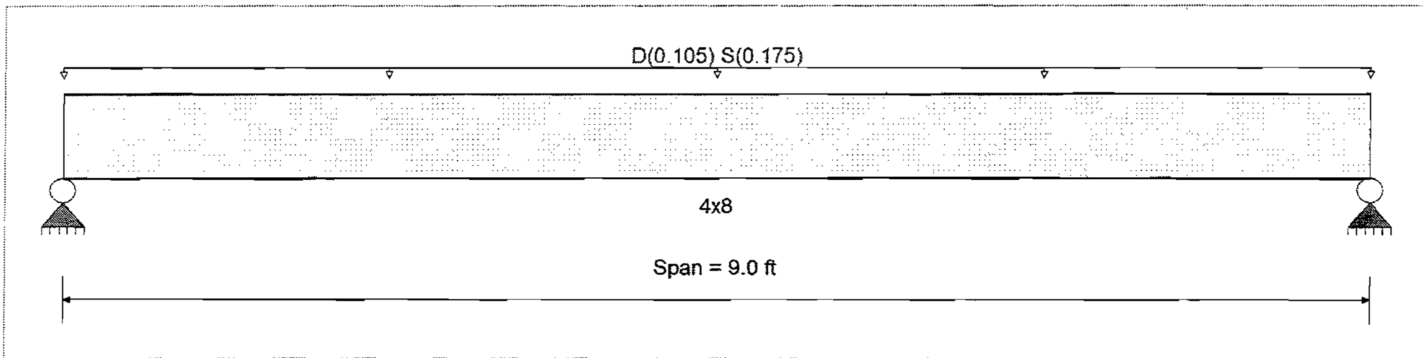
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.825: 1**	Maximum Shear Stress Ratio	=	**0.313 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,109.54 psi		=	64.70 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.409 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.146 in	Ratio =	739 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.234 in	Ratio =	461 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.260	1.260
Overall MINimum	0.788	0.788
D Only	0.473	0.473
+D+L	0.473	0.473
+D+S	1.260	1.260
+D+0.750L	0.473	0.473
+D+0.750L+0.750S	1.063	1.063
+0.60D	0.284	0.284
S Only	0.788	0.788

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Wood Beam

DESCRIPTION: 20. Floor beam at Master Shower

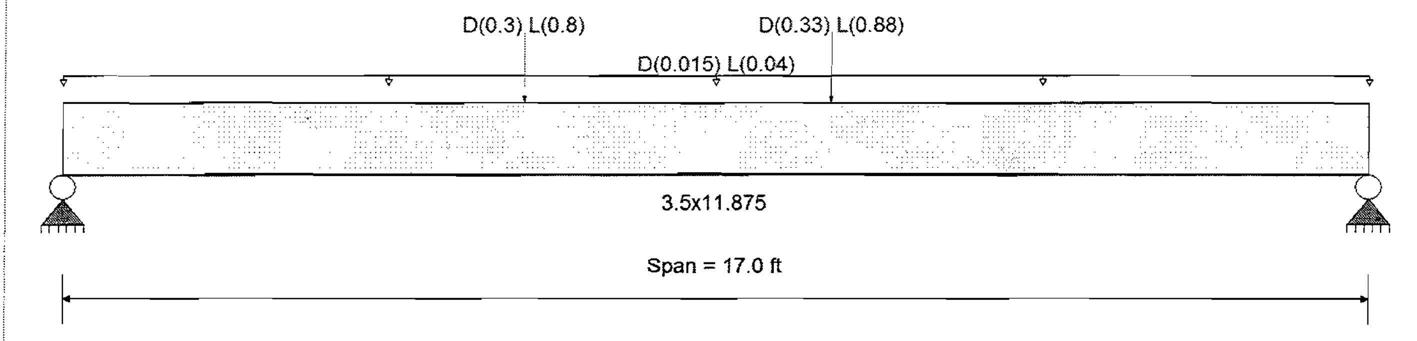
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2600 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	2600 psi	Ebend- xx	1900 ksi
		Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285 psi		
		Ft	1555 psi	Density	42.01 pcf

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Point Load : D = 0.30, L = 0.80 k @ 6.0 ft
Point Load : D = 0.330, L = 0.880 k @ 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.540**: 1	Maximum Shear Stress Ratio	=	**0.206** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	1,404.06psi		=	58.69 psi
	=	2,600.00psi		=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.989ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.378 in	Ratio =	539 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.520 in	Ratio =	392 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.678	1.568
Overall MINimum	1.220	1.140
D Only	0.458	0.428
+D+L	1.678	1.568
+D+S	0.458	0.428
+D+0.750L	1.373	1.283
+D+0.750L+0.750S	1.373	1.283
+0.60D	0.275	0.257
L Only	1.220	1.140
S Only		

Maximum Load For 6x6 DF#1 Wood Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 1245 \cdot psi$

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

$F'_c := C_p \cdot F''_c$ $F'_c = 761 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 23015 \cdot lb$ (Maximum post Capacity)

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.76$

Maximum Load For 6x6 HF#2 Treated Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 813 \cdot psi$

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

$F'_c := C_p \cdot F''_c$ $F'_c = 389 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 11760 \cdot lb$ (Maximum post Capacity)

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.85$

Maximum Load For 3-2x6 HF Stud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 934 \cdot psi$

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.71$

$F'_c := C_p \cdot F''_c$ $F'_c = 626 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 15486 \cdot lb$ (Maximum post Capacity)

Maximum Load For 2-2x6 HF Stud Built up Wood Post $psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 934 \cdot psi$

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.71$

$F'_c := C_p \cdot F''_c$ $F'_c = 626 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 10324 \cdot lb$ (Maximum post Capacity)

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PROJECT : **Marbella Residence**

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

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

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 378 \cdot psi$

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.38$

$F'_c := C_p \cdot F''_c$ $F'_c = 336 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 5299 \cdot lb$ (Maximum post Capacity)

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

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 9 \cdot ft$

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2}$ $F_{CE} = 378 \cdot psi$

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.38$

$F'_c := C_p \cdot F''_c$ $F'_c = 336 \cdot psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 3533 \cdot lb$ (Maximum post Capacity)

Maximum Load For 4x4 HF#2 Treated Post

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

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

$E' := 1235000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

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 Garage Walls
Dsgnr:
Description....

Date: 2 JAN 2017

This Wall in File: E:\My Documents\Drawings & Calcs\Retaining Walls\cantilever walls.RPX

Cantilevered Retaining Wall

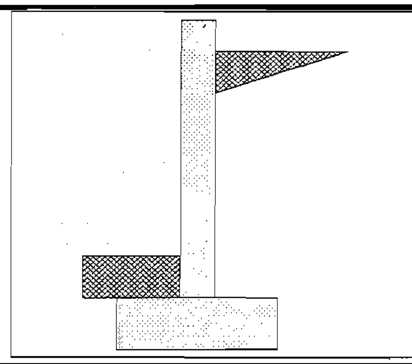
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	4.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	8.00 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	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

Surcharge Over Heel	=	40.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

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 (Service 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

Axial Load Applied to Stem

Axial Dead Load	=	100.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	2.08	OK
Sliding	=	1.71	OK
Total Bearing Load	=	1,169 lbs	
...resultant ecc.	=	5.09 in	
Soil Pressure @ Toe	=	1,050 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,470 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	4.2 psi	OK
Footing Shear @ Heel	=	4.8 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	470.3 lbs	
less 100% Passive Force	= -	337.5 lbs	
less 100% Friction Force	= -	467.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

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

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
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		=	6.00
Rebar Size		=	# 4
Rebar Spacing		=	18.00
Rebar Placed at		=	Center
Design Data			
fb/FB + fa/Fa		=	0.652
Total Force @ Section			
Service Level	lbs	=	
Strength Level	lbs	=	529.5
Moment....Actual			
Service Level	ft-#	=	
Strength Level	ft-#	=	760.2
Moment.....Allowable		=	1,165.0
Shear.....Actual			
Service Level	psi	=	
Strength Level	psi	=	14.7
Shear.....Allowable	psi	=	82.2
Anet (Masonry)	in2	=	
Rebar Depth 'd'	in	=	3.00
Masonry Data			
f'm	psi	=	
Fs	psi	=	
Solid Grouting		=	
Modular Ratio 'n'		=	
Wall Weight	psf	=	75.0
Short Term Factor		=	
Equiv. Solid Thick.		=	
Masonry Block Type		=	Medium Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	3,000.0
Fy	psi	=	40,000.0

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Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0945 in2/ft		
(4/3) * As :	0.126 in2/ft	Min Stem T&S Reinf Area 0.648 in2	
200bd/fy : 200(12)(3)/40000 :	0.18 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.144 in2/ft	
0.0018bh : 0.0018(12)(6) :	0.1296 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1296 in2/ft	#4@ 16.67 in	#4@ 33.33 in
Provided Area :	0.1333 in2/ft	#5@ 25.83 in	#5@ 51.67 in
Maximum Area :	0.7315 in2/ft	#6@ 36.67 in	#6@ 73.33 in

Footing Data

Toe Width	=	0.92 ft
Heel Width	=	1.42
Total Footing Width	=	2.33
Footing Thickness	=	10.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.92 ft

f'c = 3,000 psi Fy = 40,000 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	=	1,470	0 psf
Mu' : Upward	=	6,391	59 ft-#
Mu' : Downward	=	1,199	312 ft-#
Mu: Design	=	433	253 ft-#
Actual 1-Way Shear	=	4.21	4.82 psi
Allow 1-Way Shear	=	43.82	43.82 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	0.50	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 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

	.....OVERTURNING.....		
Item	Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	408.8	1.61	658.7
HL Act Pres (be water tbl)			
Hydrostatic Force			
Buoyant Force =			
Surcharge over Heel =	61.5	2.42	148.7
Surcharge Over Toe =			
Adjacent Footing Load =			
Added Lateral Load =			
Load @ Stem Above Soil =			
=			
Total =	470.3	O.T.M. =	807.3

Resisting/Overturning Ratio = **2.08**
Vertical Loads used for Soil Pressure = 1,169.0 lbs

	.....RESISTING.....		
	Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)	403.3	1.87	756.0
Soil Over HL (bel. water tbl)		1.87	756.0
Watre Table			
Sloped Soil Over Heel =			
Surcharge Over Heel =	36.7	1.87	68.7
Adjacent Footing Load =			
Axial Dead Load on Stem =	100.0	1.17	116.7
* Axial Live Load on Stem =			
Soil Over Toe =		0.46	
Surcharge Over Toe =			
Stem Weight(s) =	337.5	1.17	393.7
Earth @ Stem Transitions =			
Footing Weight =	291.6	1.17	340.2
Key Weight =		0.92	
Vert. Component =			
Total =	1,169.0 lbs	R.M.=	1,675.3

* 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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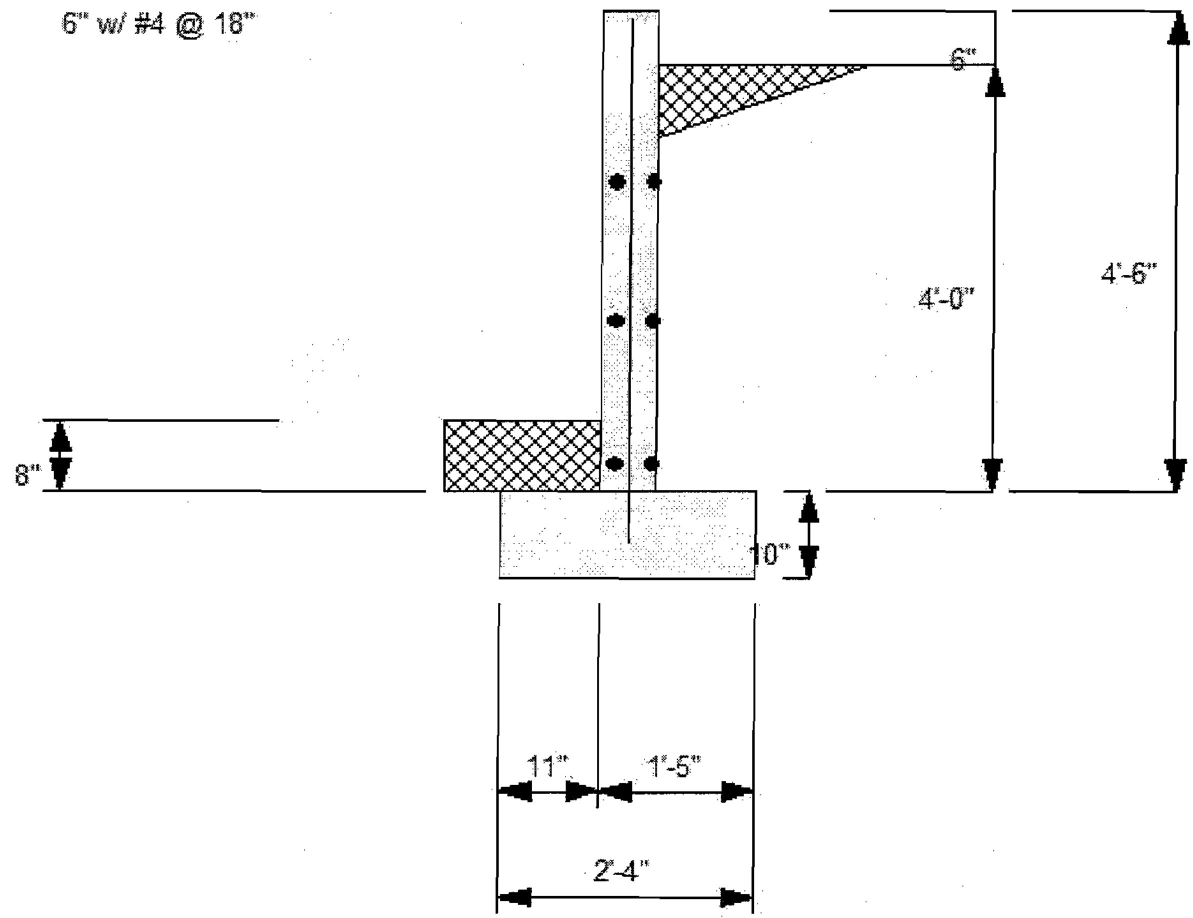
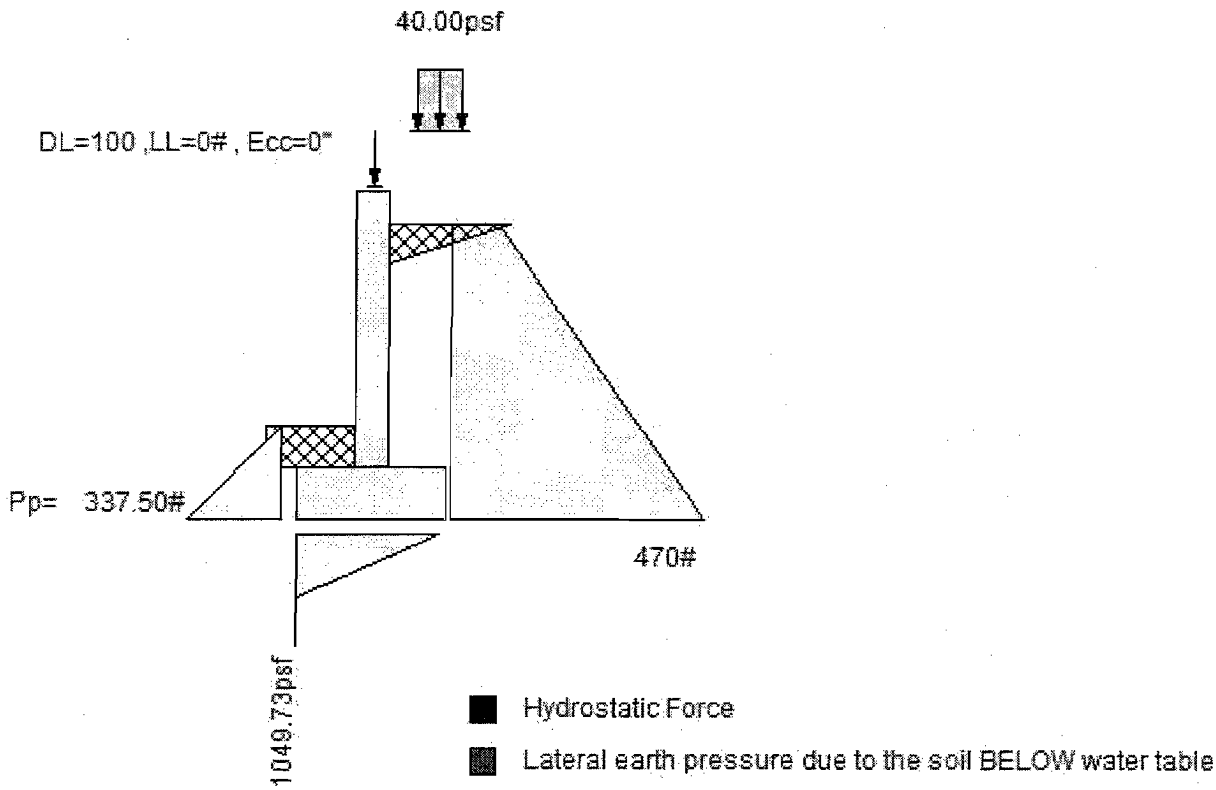
Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13



Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13



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Project Name/Number : cantilever wa
Title **6ft Stem**
Dsgnr: **Mark Myers, PE**
Description....

Page : 1
Date: 28 MAR 2016

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Cantilevered Retaining Wall

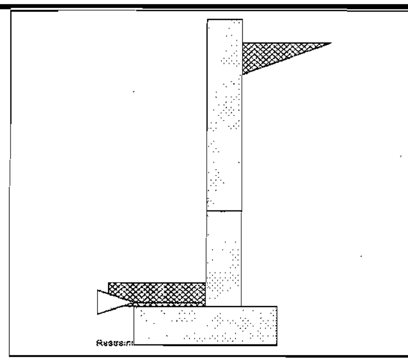
Code: IBC 2012,ACI 318-11,ACI 530-11

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		

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

Axial Load Applied to Stem

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

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 2012,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

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Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11

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 0.000 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.000 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@ 0.00 in	#4@ 0.00 in
Provided Area :	0.2 in2/ft	#5@ 0.00 in	#5@ 0.00 in
Maximum Area :	0.5419 in2/ft	#6@ 0.00 in	#6@ 0.00 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 0.000 in2	
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.000 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@ 0.00 in	#4@ 0.00 in
Provided Area :	0.24 in2/ft	#5@ 0.00 in	#5@ 0.00 in
Maximum Area :	0.5419 in2/ft	#6@ 0.00 in	#6@ 0.00 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	0.00	in2
Min footing T&S reinf Area per foot	0.00	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 0.00 in	#4@ 0.00 in
#5@ 0.00 in	#5@ 0.00 in
#6@ 0.00 in	#6@ 0.00 in

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Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... 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

Item		.....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

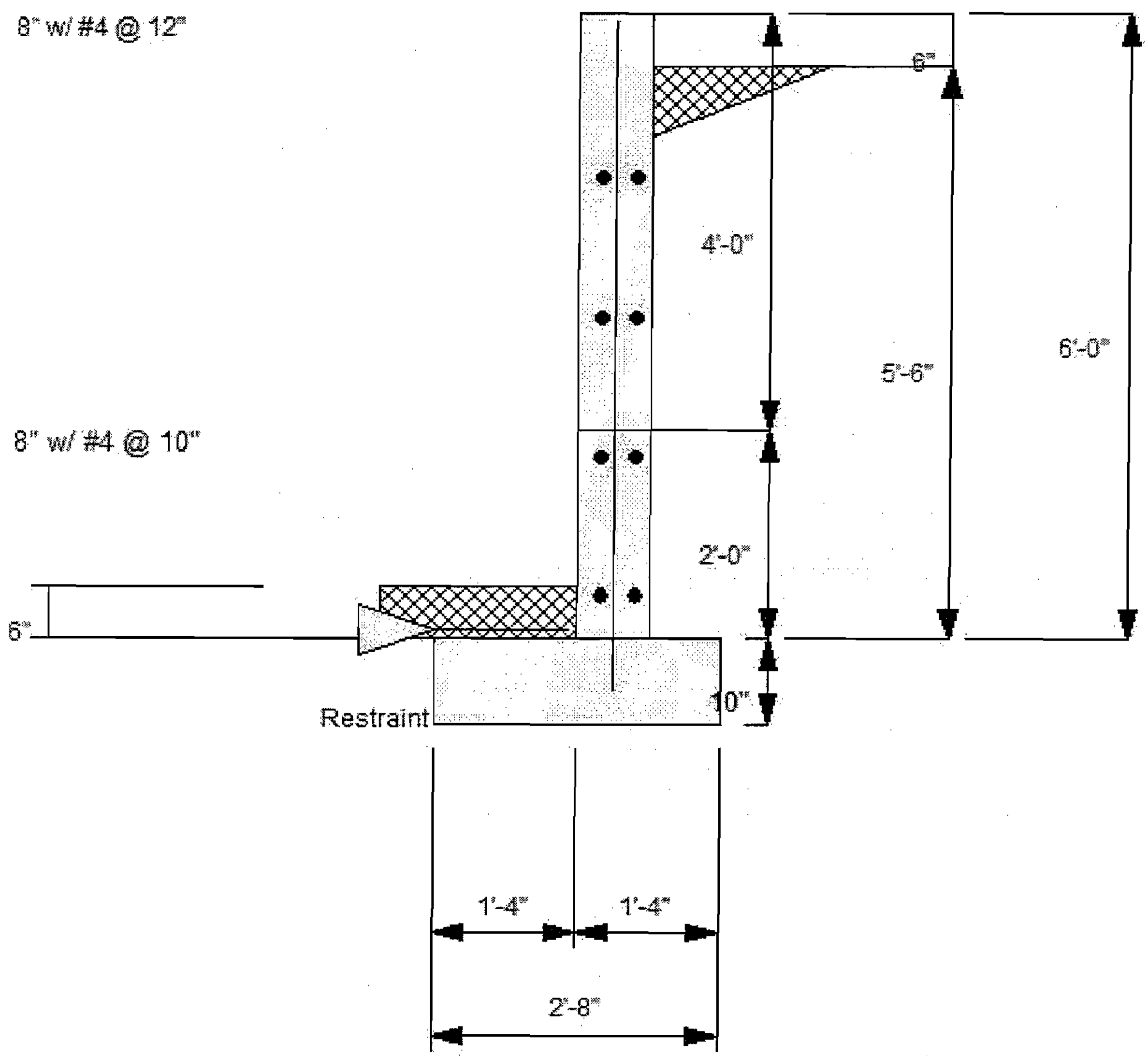
(Deflection due to wall bending not considered)

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.

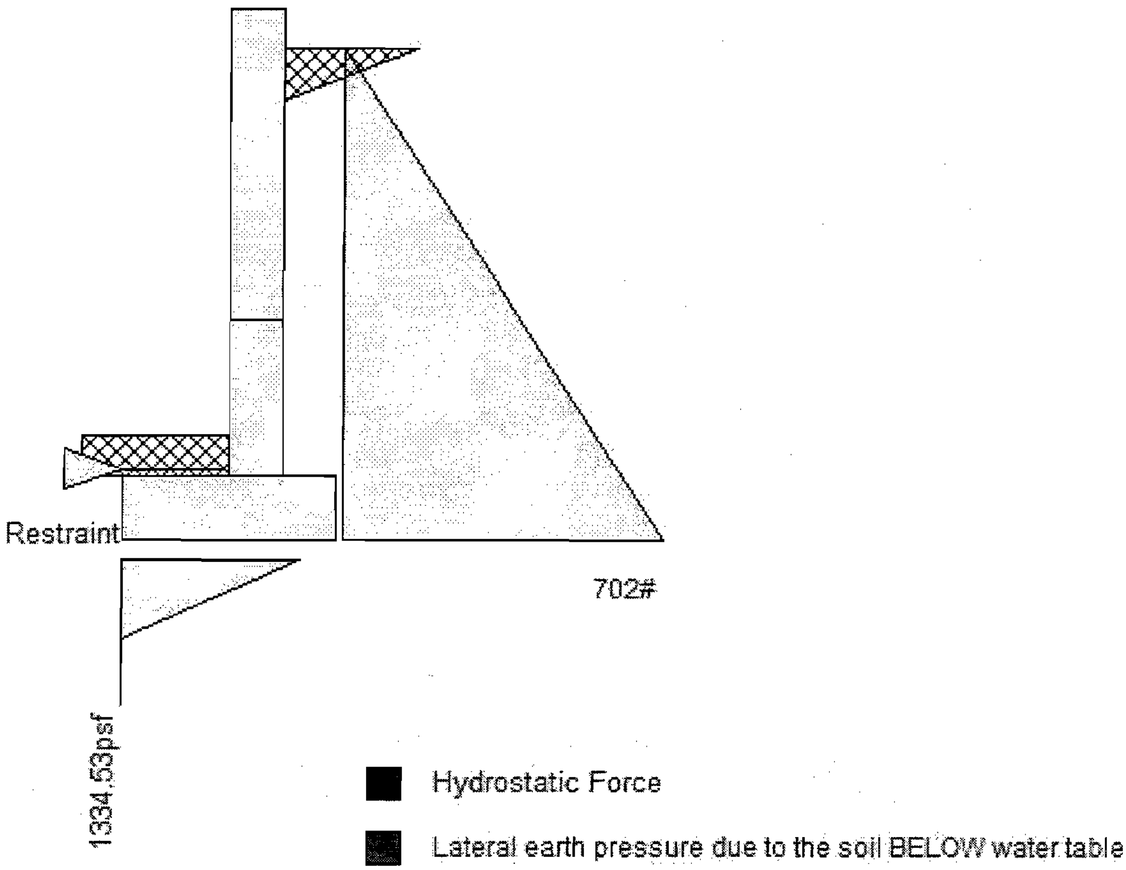
Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11



Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11



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Cantilevered Retaining Wall

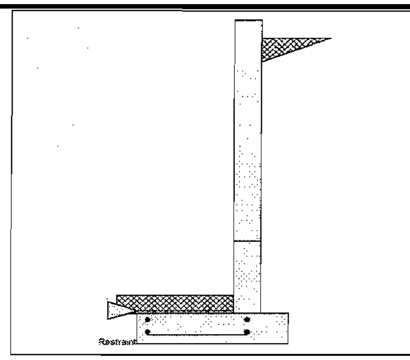
Code: IBC 2012,ACI 318-11,ACI 530-11

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 2012,ACI
Dead Load	1.400
Live Load	1.700
Earth, H	1.700
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd Stem OK	Bottom 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

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Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11

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 2012,ACI 318-11,ACI 530-11

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.....

		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

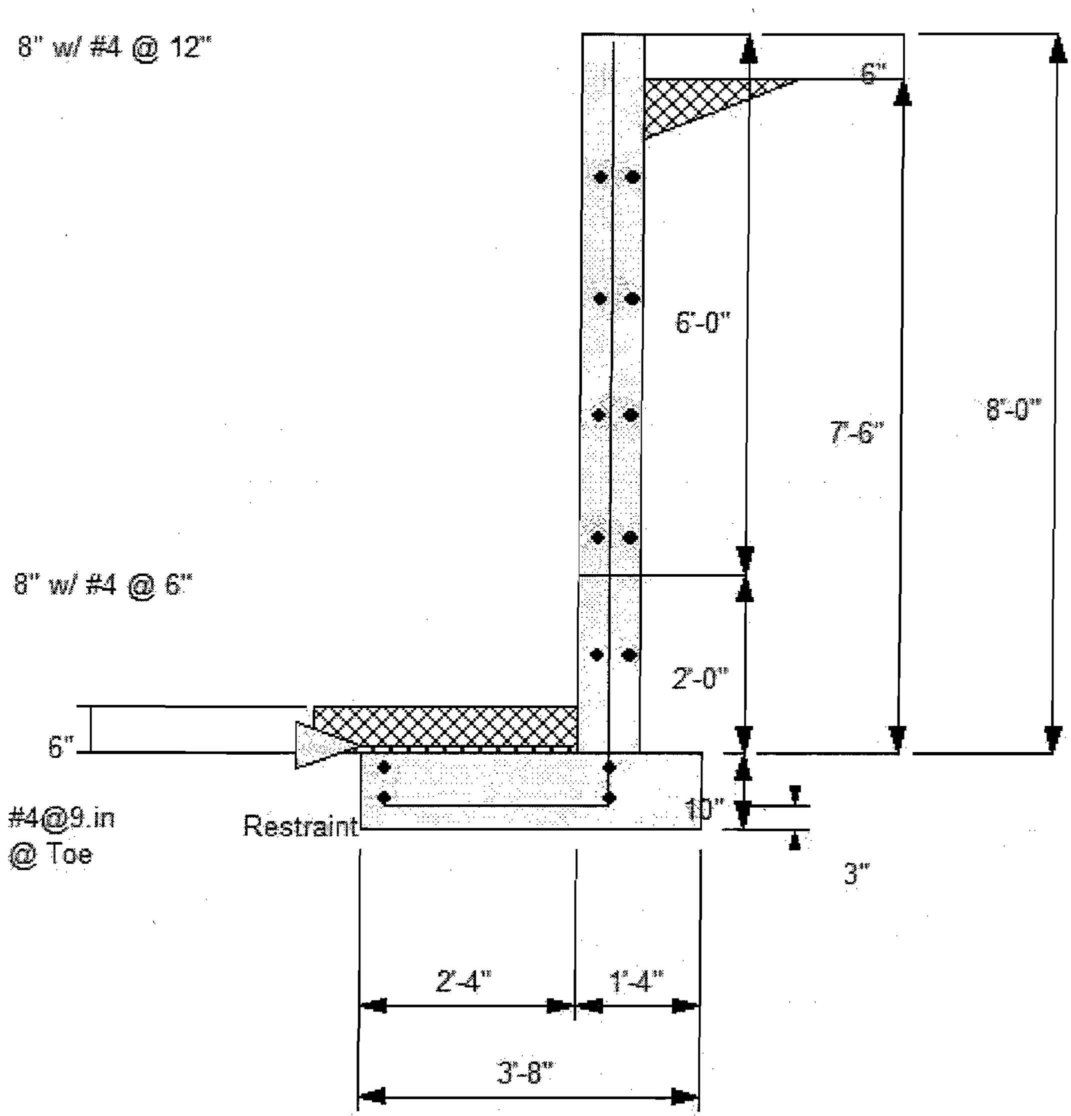
(Deflection due to wall bending not considered)

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.

Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11



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Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11