MYERS ENGINEERING

Lateral & Gravity Calculations

Digitally signed by Mark Myers
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Project: Plan M4551A3F-0
4024 85th Avenue SE
Mercer Island, WA

April 21, 2025

2021 INTERNATIONAL BUILDING CODE
100 MPH BASIC WIND, EXPOSURE B, K_{zt} = 1.00
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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DESIGN LOADS:

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

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

WOODS :

WOOD TYPE:

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

GLUED-LAMINATED (GLB) BEAM & HEADER.
F_b=2,400 PSI, F_v=165 PSI, F_c (Perp) =650 PSI, E=1,800,000 PSI.

PARALLAM (PSL) 2.0E BEAM & HEADER.
F_b=2,900 PSI, F_v=290 PSI, F_c (Perp) =750 PSI, E=2,000,000 PSI.

MICROLAM (LVL) 1.9E BEAM & HEADER
F_b=2,600 PSI, F_v=285 PSI, F_c (Perp) =750 PSI, E=1,900,000 PSI.

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD
F_b=1,700 PSI, F_v=400 PSI, F_c (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.

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LATERAL ANALYSIS :
BASED ON 2021 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 C (Per Geotech)

SEISMIC DESIGN:
SEISMIC DESIGN BASED ON 2021 IBC Section 1613.1
LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT ABOVE GRADE.

Seismic Design Data:

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

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

$S_s := 1.414$ $S_1 := 0.492$ $S_{ms} := 1.696$ $S_{m1} := 0.737$

Equation 11.4-3 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.13$ Equation 11.4-4 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.49$

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

Plan Area for Each Level:

$A_1 := 1.12\ (3086) \cdot ft^2$ $A_{2a} := 2442 \cdot ft^2$ $A_{2b} := 1.12\ (555) \cdot ft^2$

(Upper Roof) (Upper Floor) (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2 \cdot (40 \cdot ft) + 2 \cdot (69 \cdot ft)$ $P_2 := 2 \cdot (44 \cdot ft) + 2 \cdot (69 \cdot ft)$

(Upper Floor) (Main Floor)

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

Weight of Structure at Each Level:

Story Weight at Upper Floor:

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

Story Weight at Main Floor:

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

$W := w_1 + w_2 = 134902.80\ lb$

Roof Adjustment:

Slope	*Factor*
4	1.05
5	1.08
6	1.12
7	1.16
8	1.20
9	1.25
10	1.3
11	1.36
12	1.41

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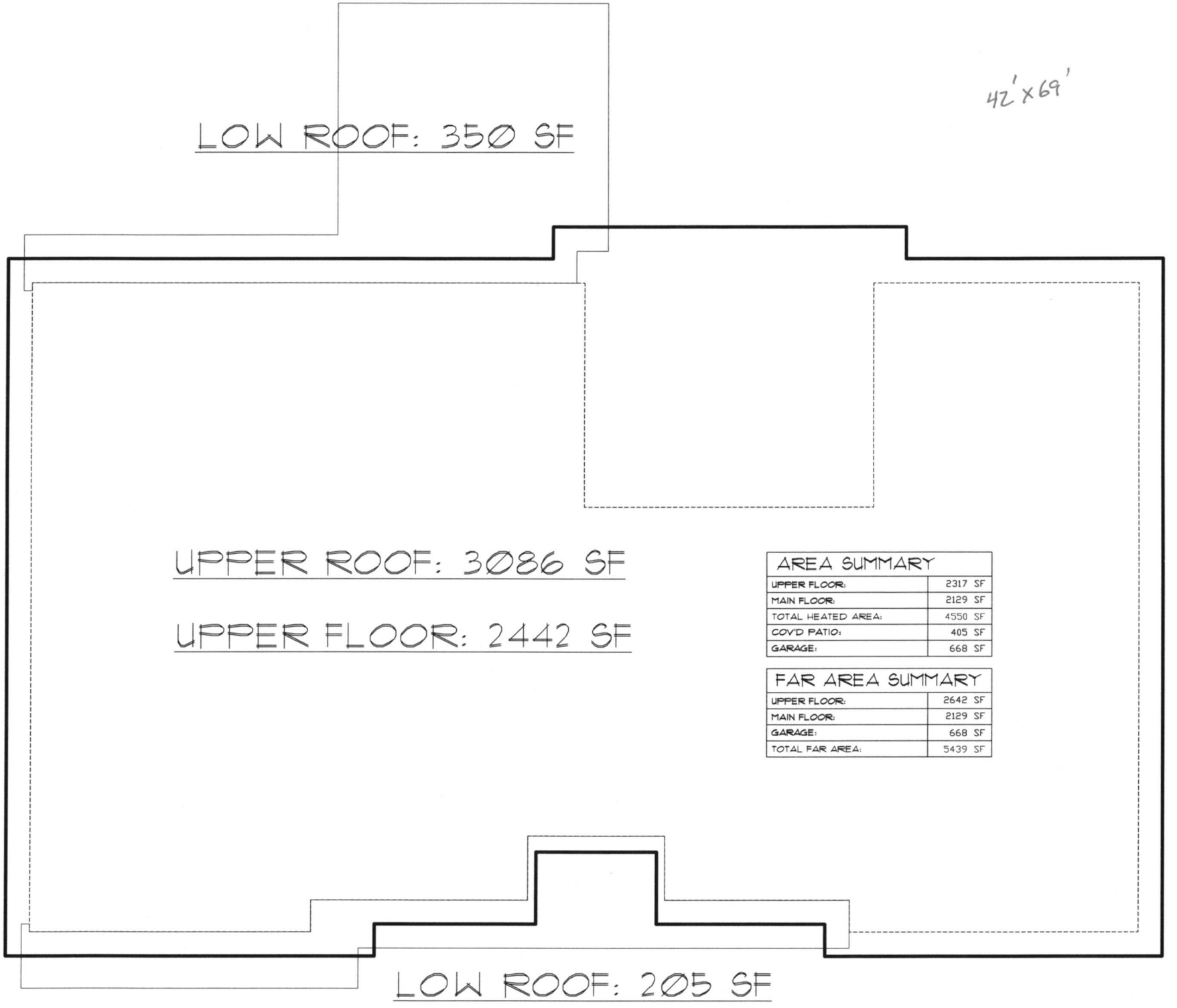

AREA SUMMARY	
UPPER FLOOR:	2317 SF
MAIN FLOOR:	2129 SF
TOTAL HEATED AREA:	4550 SF
COV'D PATIO:	405 SF
GARAGE:	668 SF

FAR AREA SUMMARY	
UPPER FLOOR:	2642 SF
MAIN FLOOR:	2129 SF
GARAGE:	668 SF
TOTAL FAR AREA:	5439 SF

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Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $h_n := 26$ (Structural Height per ASCE 7-16 Sect. 11.2)

$T_a := C_t \cdot h_n^{\chi} = 0.23$ (ASCE 7-16 Eq. 12.8-7) $T_L := 6$ (per ASCE 7-16 Fig. 22-14)

T_a is less than T_L, therefore Cs need not exceed: $\frac{S_{D1}}{\left(\frac{R}{I_e}\right) \cdot T_a} = 0.33$ (ASCE 7-16 Eq. 12.8-3)

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

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

Total Base Shear: $V_E := C_s \cdot W = 23466.17\ \textbf{lb}$

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

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

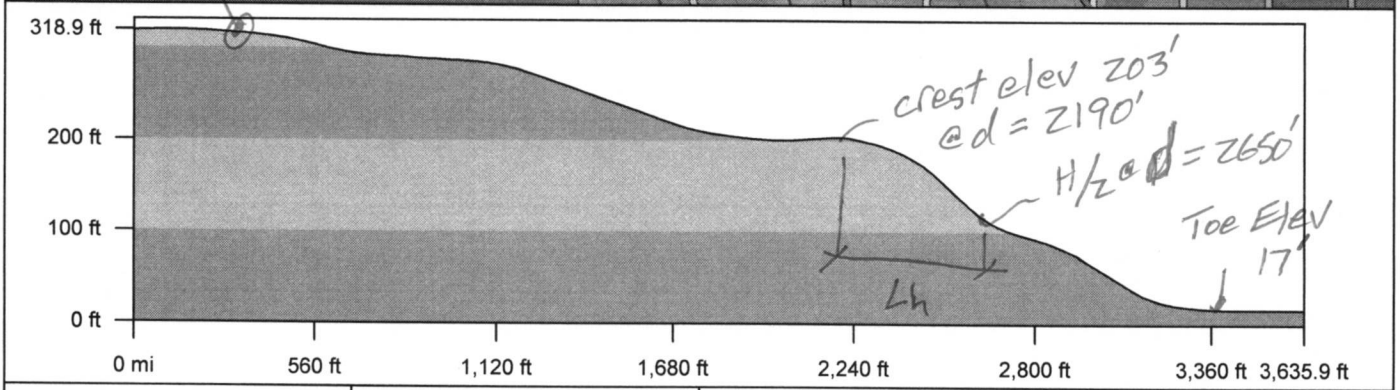
$h_1 := 20 \cdot \textbf{ft}$ $h_2 := 10 \cdot \textbf{ft}$ (Height from base to level x)

$C_{v1} := \frac{\left(w_1 \cdot h_1\right)}{\left(w_1 \cdot h_1 + w_2 \cdot h_2\right)} = 0.64$ $F_1 := C_{v1} \cdot V_E = 15039.75\ \textbf{lb}$ Story Shear at Upper Floor

$C_{v2} := \frac{\left(w_2 \cdot h_2\right)}{\left(w_1 \cdot h_1 + w_2 \cdot h_2\right)} = 0.36$ $F_2 := C_{v2} \cdot V_E = 8426.42\ \textbf{lb}$ Story Shear at Main Floor

DeLorme

DeLorme Topo USA® 7.0



Lin Dist: 3,611.3 ft	Terr Dist: 3,635.9 ft	Elev Gain: -301.8 ft	Avg Grade: 8
Climb Elev: 4.0 ft	Desc Elev: 305.8 ft	Max. Elev: 318.9 ft	Min. Elev: 16.3 ft
Climb Dist: 435.7 ft	Desc Dist: 3,200.2 ft		

TN

Scale 1 : 8,800

1" = 733.3 ft

Data Zoom 14-5

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WIND DESIGN

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

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

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 26 \cdot ft$ Mean Roof Height as per Sect. 26.2

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

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

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

$x := 1932 \cdot ft$ $H := 186 \cdot ft$ $L_h := 460 \cdot ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$K_1 := 0.75 \cdot \left(\frac{H}{L_h}\right) = 0.30$ $K_2 := \left(1 - \frac{x}{\mu \cdot L_h}\right) = -0.05$ $K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.87$ $K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 0.97$

Therefore: $\boxed{K_{zt}} := 1.00$

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

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

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

Velocity Pressure Exposure Coefficient (Table 26.10-1):

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

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

$K_{z1} := 2.01 \cdot \left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.62$ $K_{z2} := 2.01 \cdot \left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.57$ $K_h := 2.01 \cdot \left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.67$

External Pressure Coefficients w/ Roof Pitch = 5.56/12 (25 degrees) Front to Back & 10/12 (40 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 42 \cdot ft$ $B_{fb} := 69 \cdot ft$ $\frac{L_{fb}}{B_{fb}} = 0.61$ $\frac{h}{L_{fb}} = 0.62$

$C_{pf1} := 0.8$ Windward Wall

$C_{pf2} := 0.152$ Windward Roof

$C_{pf3} := -0.6$ Leeward Roof

$C_{pf4} := -0.5$ Leeward Wall

Side to Side:

$L_{ss} := 69 \cdot ft$ $B_{ss} := 42 \cdot ft$ $\frac{L_{ss}}{B_{ss}} = 1.64$ $\frac{h}{L_{ss}} = 0.38$

$C_{ps1} := 0.8$ Windward Wall

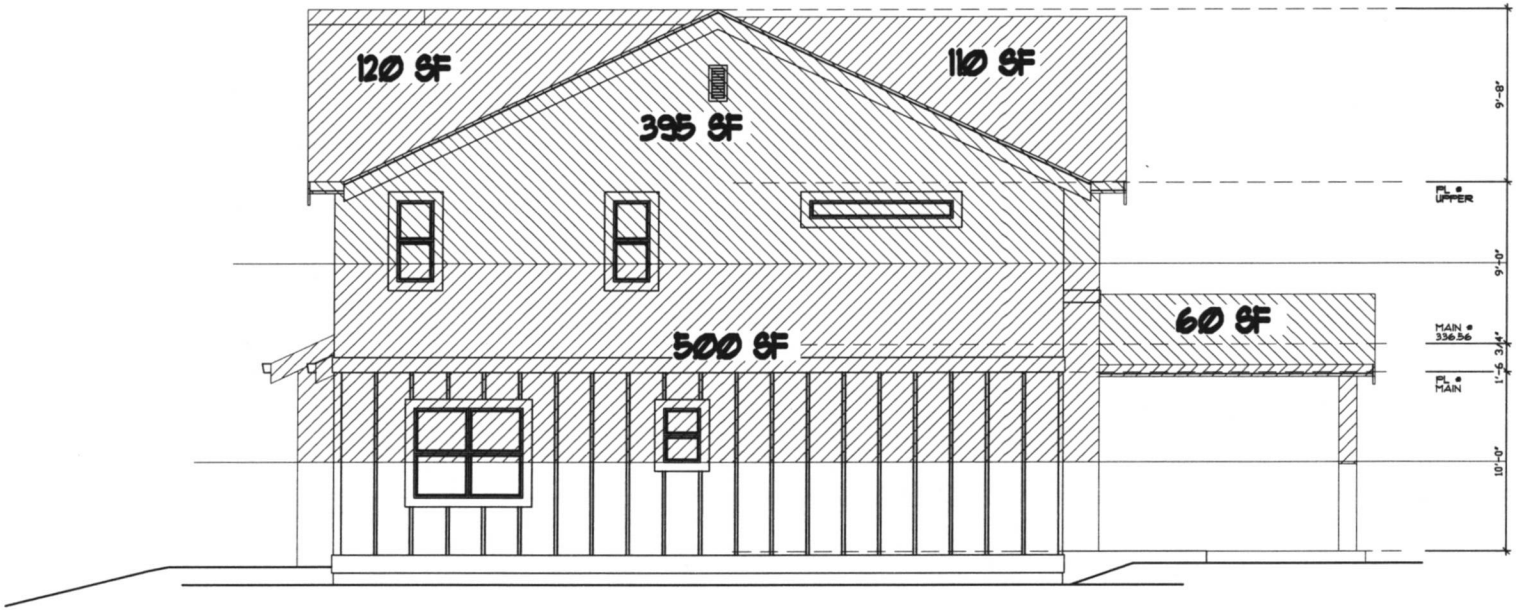
$C_{ps2} := 0.374$ Windward Roof

$C_{ps3} := -0.6$ Leeward Roof

$C_{ps4} := -0.372$ Leeward Wall

FRONT (W) ELEVATION

SCALE: 1/4" • 1'-0"



RIGHT (S) ELEVATION

SCALE: 1/4" • 1'-0"

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.58$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 12.51$

$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 14.63$

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

Windward Wall both directions $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 9.23 \ \frac{lb}{ft^2}$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 8.50 \ \frac{lb}{ft^2}$

Front to Back Pressures:

Windward Roof $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 1.89 \ \frac{lb}{ft^2}$

Leeward Roof $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -7.46 \ \frac{lb}{ft^2}$

Leeward Wall $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -6.22 \ \frac{lb}{ft^2}$

Side to Side Pressures:

Windward Roof $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 4.65 \ \frac{lb}{ft^2}$

Leeward Roof $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -7.46 \ \frac{lb}{ft^2}$

Leeward Wall $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -4.63 \ \frac{lb}{ft^2}$

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 8psf at roof & 16psf at walls over projected vertical plane per ASCE 7-16 Sec. 27.1-5:

$p_{wr1} - p_{lr1} = 9.35 \ \frac{lb}{ft^2}$ $p_{ww1} - p_{lw1} = 15.45 \ \frac{lb}{ft^2}$ $p_{ww2} - p_{lw1} = 14.72 \ \frac{lb}{ft^2}$

$p_{wr2} - p_{lr2} = 12.12 \ \frac{lb}{ft^2}$ $p_{ww1} - p_{lw2} = 13.86 \ \frac{lb}{ft^2}$ $p_{ww2} - p_{lw2} = 13.13 \ \frac{lb}{ft^2}$

Wind Pressure at Upper Roof (Front to Back):

$V_{1W} := (p_{wr1} - p_{lr1}) \cdot 405 \cdot ft^2 + (16 \ psf) \cdot 600 \cdot ft^2 = 13388.43 \ lb$

Wind Pressure at Main Floor (Front to Back):

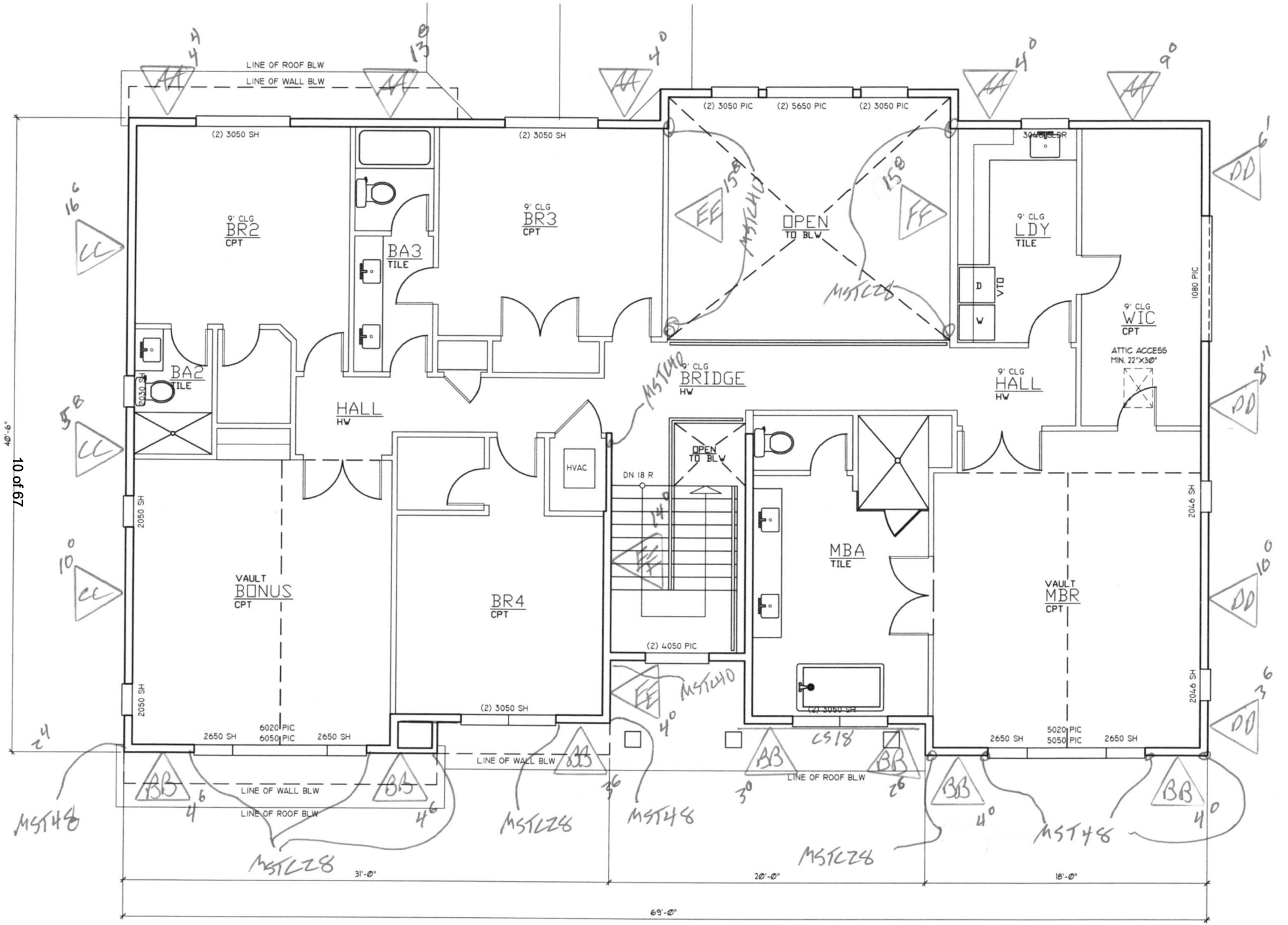
$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 0 \cdot ft^2 + (16 \ psf) \cdot 850 \cdot ft^2 = 13600.00 \ lb$

Wind Pressure at Upper Roof (Side to Side):

$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 230 \cdot ft^2 + (16 \ psf) \cdot 395 \cdot ft^2 = 9106.59 \ lb$

Wind Pressure at Main Floor (Side to Side):

$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 60 \cdot ft^2 + (16 \ psf) \cdot 500 \cdot ft^2 = 8726.94 \ lb$


LINE OF ROOF BLW
LINE OF WALL BLW
(2) 3050 SH
(2) 3050 SH
(2) 3050 PIC
(2) 5650 PIC
(2) 3050 PIC
9' CLG
BR2
CPT
BA3
TILE
9' CLG
BR3
CPT
OPEN
TO BLW
9' CLG
LDY
TILE
D
W
VTD
9' CLG
WIC
CPT
ATTIC ACCESS
MIN. 22"X30"
1080 PIC
BA2
TILE
2050 SH
HALL
HW
9' CLG
BRIDGE
HW
9' CLG
HALL
HW
HVAC
DN 18 R
OPEN
TO BLW
MBA
TILE
VAULT
BONUS
CPT
BR4
CPT
VAULT
MBR
CPT
2046 SH
2046 SH
2050 SH
2050 SH
(2) 4050 PIC
(2) 3050 SH
(2) 3050 SH
2650 SH
6020 PIC
6050 PIC
2650 SH
2650 SH
5020 PIC
5050 PIC
2650 SH
LINE OF WALL BLW
LINE OF ROOF BLW
LINE OF WALL BLW
LINE OF ROOF BLW
40'-6"
31'-Ø"
20'-Ø"
18'-Ø"
69'-Ø"

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WALL AA:

Story Shear due to Wind: $V_{3W} = 9106.59\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

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

Distance between shear walls: $L_1 := 40 \cdot ft$

Shear Wall Length: $Laa := \left(4.33\left(1.25 - 0.125\left(\frac{9}{4.33}\right)\right) + 13.67 + 2 \cdot 4\left(1.25 - 0.125\left(\frac{9}{4}\right)\right) + 9\right) \cdot ft = 34.71\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

Wind Force: $vaa := \frac{\frac{0.6 \cdot V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Laa}$

Seismic Force: $\rho := 1.3$ $E_{aa} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1}{2}}{Laa}$

$vaa = 78.71\ \frac{lb}{ft}$ $\frac{vaa}{C_o} = 78.71\ \frac{lb}{ft}$

$E_{aa} = 197.16\ \frac{lb}{ft}$ $\frac{E_{aa}}{C_o} = 197.16\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{aa} := 4 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRaa := \frac{W_{aa} \cdot L_{aa}}{2} = 540.00\ lb$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 708.43\ lb$

$CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 1774.48\ lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 384.43\ lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRaa = 1535.96\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 2.07\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 0.83\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vaa} = 17.48\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{aa}} = 6.98\ ft$

5/8" dia. anchors @ 72" o.c.

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WALL BB:

Story Shear due to Wind: $V_{3W} = 9106.59\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

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

Distance between shear walls: $L_1 := 40 \cdot ft$

Shear Wall Length: $Lbb := \left(2 \cdot 4.5 + 3.5 \left(1.25 - 0.125 \left(\frac{9}{3.5}\right)\right) + 2 \cdot 4 \left(1.25 - 0.125 \left(\frac{9}{4}\right)\right) + 3 + 2.5\right) \cdot ft = 25.50\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

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

Seismic Force: $\rho := 1.3$ $E_{bb} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1}{2}}{Lbb}$

$vbb = 107.14\ \frac{lb}{ft}$ $\frac{vbb}{C_o} = 107.14\ \frac{lb}{ft}$

$E_{bb} = 268.36\ \frac{lb}{ft}$ $\frac{E_{bb}}{C_o} = 268.36\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{bb} := 4.42 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2} = 265.20\ lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} = 964.23\ lb$

$CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} = 2415.21\ lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 805.11\ lb$

$HDFbb_s := CFbb_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRbb = 2298.07\ lb$

Simpson MST48 strap to wall or MSTC28 to rim beam

See FTAO Calc for wall at Master Bath

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vbb} = 1.52\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.61\ ft$

16d Sinkers @ 6" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vbb} = 12.84\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{bb}} = 5.13\ ft$

5/8" dia. anchors @ 72" o.c.

Force Transfer Around Openings Calculator

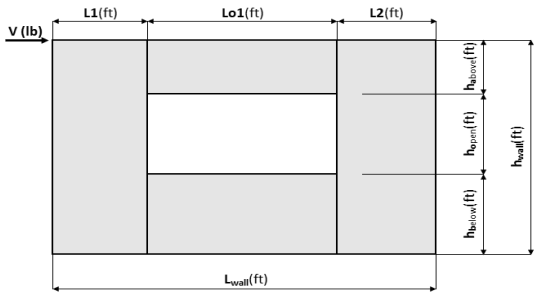
ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2018 IBC
Designer:	Mark Myers, PE
Client:	NWEB
Project:	4024 85th AVE SE
Wall Line:	BB

Date: 4/18/2025



Shear Wall Calculation Variables

V	1485 lbf	Opening 1	
L1	3.00 ft	h_a	1.00 ft
L2	2.50 ft	h_o	5.00 ft
h_{wall}	9.00 ft	h_b	3.00 ft
L_{wall}	11.50 ft	Lo1	6.00 ft

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	1.67	N/A
P2=h_o/L2=	2.00	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1162 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) = 291 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1743 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 951 lbf

F2 = O1(L2)/(L1+L2) = 792 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.27 ft

T2 = (L2*Lo1)/(L1+L2) = 2.73 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 270 plf

v2 = (V/L)(T2+L2)/L2 = 270 plf

Check v1*L1+v2*L2=V? 1485 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 810 lbf

R2 = v2*L2 = 675 lbf

8. Difference corner force + resistance

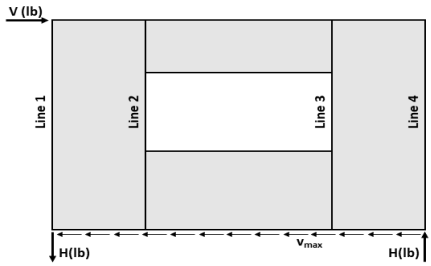
R1-F1 = -141 lbf

R2-F2 = -117 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -47 plf

vc2 = (R2-F2)/L2 = -47 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(h_a+h_b)+v1(h_o)=H?		-188	1350	1162 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	1162	-188	1350	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	1162	-188	1350	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		-188	1350	1162 lbf

Design Summary*

Req. Sheathing Capacity	291 plf	**4-Term Deflection**	0.416 in.	**3-Term Deflection**	0.453 in.
Req. Strap Force	951 lbf	**4-Term Story Drift %**	0.015 %	**3-Term Story Drift %**	0.017 %
Req. HD Force (H)	1162 lbf				
Req. Shear Wall Anchorage Force (v_{max})	129 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2120 (lbf)

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Code:	2018 IBC	**Date:**	4/18/2025
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Project:	4024 85th AVE SE		
Wall Line:	BB		

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: Doug-Fir
E: 1.40E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	3460	3460	(lbf)
HD Deflection:	0.125	0.125	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	385	385	385	385	(plf)
E:	1.40E+06	1.40E+06	1.40E+06	1.40E+06	(psi)
h:	9.00	6.00	6.00	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					($in.^2$)
A:	16.5	16.5	16.5	16.5	($in.^2$)
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	128	128	128	128	(plf)
e_n:	0.0106	0.0106	0.0106	0.0106	(in.)
b:	3.00	3.00	2.50	2.50	(ft)
HD Capacity:	3460	3460	3460	3460	(lbf)
HD Defl:	0.125	0.125	0.125	0.125	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.032	0.042	0.071	0.376	0.010	0.028	0.048	0.167
		Sum	0.521			Sum	0.252
Pier 2 (left)				Pier 2 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.012	0.028	0.048	0.201	0.039	0.042	0.071	0.451
		Sum	0.287			Sum	0.603

Total Defl.	
0.416	(in.)
0.0154	%drift

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2120 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: Doug-Fir
E: 1.40E+06 (psi)

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	3460	3460	(lbf)
HD Deflection:	0.125	0.125	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	385	385	385	385	(plf)
E:	1.40E+06	1.40E+06	1.40E+06	1.40E+06	(psi)
h:	9.00	6.00	6.00	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					($in.^2$)
A:	16.5	16.5	16.5	16.5	($in.^2$)
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.00	3.00	2.50	2.50	(ft)
HD Capacity:	3460	3460	3460	3460	(lbf)
HD Defl:	0.125	0.125	0.125	0.125	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener

Total

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Wall Line:	BB

Date: 4/18/2025

0.032	0.158	0.376	0.010	0.105	0.167
	Sum	0.566		Sum	0.282
Pier 2 (left)			Pier 2 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.012	0.105	0.201	0.039	0.158	0.451
	Sum	0.317		Sum	0.648

Defl.	
0.453	(in.)
0.0168	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

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PROJECT : **4024 85th AVE SE**

WALL CC:

Story Shear due to Wind: $V_{1W} = 13388.43\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 31 \cdot ft$

Shear Wall Length: $Lcc := (10 + 5.67 + 16.5) \cdot ft = 32.17\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

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

Seismic Force: $\rho := 1.3$ $E_{cc} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1}{2}}{Lcc}$

$vcc = 56.09\ \frac{lb}{ft}$ $\frac{vcc}{C_o} = 56.09\ \frac{lb}{ft}$

$E_{cc} = 95.57\ \frac{lb}{ft}$ $\frac{E_{cc}}{C_o} = 95.57\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{cc} := 5.67 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRcc := \frac{W_{cc} \cdot L_{cc}}{2} = 340.20\ lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} = 504.84\ lb$

$CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} = 860.11\ lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6 \cdot DLRcc = 300.72\ lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRcc = 709.85\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vcc} = 2.91\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 1.71\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vcc} = 24.53\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{cc}} = 14.40\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

WALL DD:

Story Shear due to Wind: $V_{1W} = 13388.43\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 18 \cdot ft$

Shear Wall Length: $Ldd := \left(6.08 + 8.92 + 10 + 3.5\left(1.25 - 0.125\left(\frac{9}{3.5}\right)\right)\right) \cdot ft = 28.25\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

Wind Force: $vdd := \frac{0.6 \cdot V_{1W} \cdot \frac{L_1}{2}}{Ldd}$

Seismic Force: $\rho := 1.3$ $E_{dd} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1}{2}}{Ldd}$

$vdd = 37.09\ \frac{lb}{ft}$ $\frac{vdd}{C_o} = 37.09\ \frac{lb}{ft}$

$E_{dd} = 63.19\ \frac{lb}{ft}$ $\frac{E_{dd}}{C_o} = 63.19\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{dd} := 3.5 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRdd := \frac{W_{dd} \cdot L_{dd}}{2} = 210.00\ lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} = 333.81\ lb$

$CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} = 568.72\ lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6 \cdot DLRdd = 207.81\ lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRdd = 475.96\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vdd} = 4.40\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 2.58\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vdd} = 37.10\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{dd}} = 21.78\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

WALL EE:

Story Shear due to Wind: $V_{1W} = 13388.43\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 20 \cdot ft$ $L_2 := 31 \cdot ft$

Shear Wall Length: $Lee := \left(15.67 + 14 + 4\left(1.25 - 0.125\left(\frac{9}{4}\right)\right)\right) \cdot ft = 33.55\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

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

$vee = 88.50\ \frac{lb}{ft}$ $\frac{vee}{C_o} = 88.50\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_{ee} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lee}$

$E_{ee} = 150.78\ \frac{lb}{ft}$ $\frac{E_{ee}}{C_o} = 150.78\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{ee} := 4 \cdot ft$

Plate Height: $Pt := 9 \cdot ft$

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

$DLRee := \frac{W_{ee} \cdot L_{ee}}{2} = 240.00\ lb$

Chord Force:

$CFee_w := \frac{vee \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}} = 796.50\ lb$

$CFee_s := \frac{E_{ee} \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}} = 1357.02\ lb$

Holdown Force:

$HDFee_w := CFee_w - 0.6 \cdot DLRee = 652.50\ lb$

$HDFee_s := CFee_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRee = 1251.02\ lb$

Simpson MSTC40 to wall or low beam & MSTC28 strap to rim beam

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vee} = 1.84\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ee}} = 1.08\ ft$

16d Sinkers @ 12" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vee} = 15.55\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ee}} = 9.13\ ft$

5/8" dia. anchors @ 72" o.c.

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WALL FF:

Story Shear due to Wind: $V_{1W} = 13388.43\ lb$

Story Shear due to Seismic: $F_1 = 15039.75\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 20 \cdot ft$ $L_2 := 18 \cdot ft$

Shear Wall Length: $Lff := (15.67) \cdot ft = 15.67\ ft$

Percent full height sheathing: $\left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 = 100.00$

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

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

Seismic Force: $\rho := 1.3$ $E_{ff} := \frac{\rho \cdot \frac{0.7 \cdot F_1}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lff}$

$vff = 141.16\ \frac{lb}{ft}$ $\frac{vff}{C_o} = 141.16\ \frac{lb}{ft}$

$E_{ff} = 240.50\ \frac{lb}{ft}$ $\frac{E_{ff}}{C_o} = 240.50\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{ff} := 15.67 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRff := \frac{W_{ff} \cdot L_{ff}}{2} = 940.20\ lb$

Chord Force:

$CFff_w := \frac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 1270.45\ lb$

$CFff_s := \frac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 2164.51\ lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6 \cdot DLRff = 706.33\ lb$

$HDFff_s := CFff_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRff = 1749.22\ lb$

Simpson MSTC28 strap to rim beam

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vff} = 1.16\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ff}} = 0.68\ ft$

16d Sinkers @ 8" o.c.

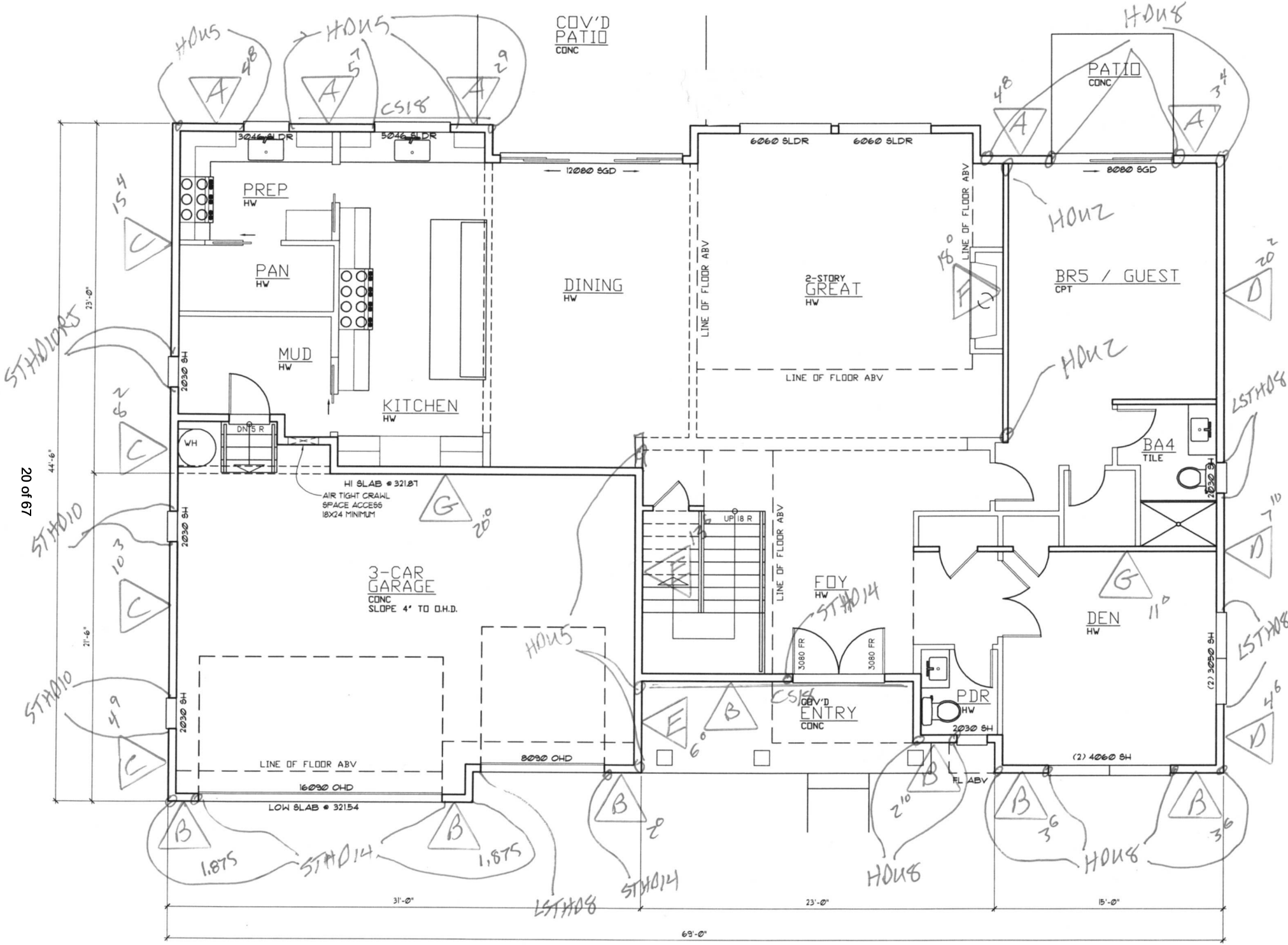
Anchor Bolt Spacing (2018 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vff} = 9.75\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ff}} = 5.72\ ft$

5/8" dia. anchors @ 66" o.c.

Mark Myers, PE 04/18/2025


COV'D PATIO
CONC
PATIO
CONC
PREP
HW
PAN
HW
MUD
HW
KITCHEN
HW
DINING
HW
2-STORY GREAT
HW
BR5 / GUEST
CPT
BA4
TILE
DEN
HW
FOY
HW
PDR
HW
COV'D ENTRY
CONC
3-CAR GARAGE
CONC
SLOPE 4" TO O.H.D.
DN'S R
UP 18 R
WH
HI SLAB @ 321.87
AIR TIGHT CRAWL SPACE ACCESS 18X24 MINIMUM
LOW SLAB @ 321.54
LINE OF FLOOR ABV
3046 SLDR
6060 SLDR
12080 SGD
8080 SGD
2030 SH
3080 FR
(2) 3050 SH
(2) 4060 SH
8030 OHD
16030 OHD
HOU5
STH010
STH014
LSTH08
CS18
31'-0"
23'-0"
15'-0"
69'-0"

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PROJECT : **4024 85th AVE SE**

WALL A:

Story Shear due to Wind: $V_{4W} = 8726.94\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 44.5 \cdot ft$

Distance between shear walls: $L_1 := 23 \cdot ft$

Shear Wall Length: $La := \left(2 \cdot 4.67 \left(1.25 - 0.125 \left(\frac{10}{4.67}\right)\right) + 5.58 + 2.75 + 3.33 \left(1.25 - 0.125 \left(\frac{10}{3.33}\right)\right)\right) \cdot ft = 20.42\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

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

$va = 200.08\ \frac{lb}{ft}$ $\frac{va}{C_o} = 200.08\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_a := \frac{E_{aa} \cdot Laa + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$E_a = 432.21\ \frac{lb}{ft}$ $\frac{E_a}{C_o} = 432.21\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_a := 3.33 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRa := \frac{W_a \cdot L_a}{2} = 299.70\ lb$

Chord Force:

$CFa_w := \frac{va \cdot L_a \cdot Pt}{C_o \cdot L_a} = 2000.80\ lb$

$CFa_s := \frac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a} = 4322.13\ lb$

$CFa_w + CFaa_w = 2709.23\ lb$

$CFa_s + CFaa_s = 6096.61\ lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 1820.98\ lb$

$HDFa_s := CFa_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRa = 4189.75\ lb$

Simpson HDU5 w/ SB5/8x24 anchor

$HDFa_w + HDFaa_w = 2205.41\ lb$

$HDFa_s + HDFaa_s = 5725.71\ lb$

Simpson HDU8 w/ SB7/8x24 anchor

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{va} = 0.82\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.38\ ft$

16d Sinkers @ 4" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{va} = 6.88\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_a} = 3.18\ ft$

5/8" dia. anchors @ 32" o.c.

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PROJECT : **4024 85th AVE SE**

WALL B:

Story Shear due to Wind: $V_{4W} = 8726.94\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 44.5 \cdot ft$

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

Shear Wall Length: $Lb := \left(2 \cdot 1.875 + 2 + 9.5 + 2.83\left(1.25 - 0.125\left(\frac{10}{2.83}\right)\right) + 2 \cdot 3.5\left(1.25 - 0.125\left(\frac{10}{3.5}\right)\right)\right) \cdot ft = 23.79\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

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

Seismic Force: $\rho := 1.3$ $E_b := \frac{E_{bb} \cdot Lbb + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Lb}$

$vb = 168.02\ \frac{lb}{ft}$ $\frac{vb}{C_o} = 168.02\ \frac{lb}{ft}$

$E_b = 365.55\ \frac{lb}{ft}$ $\frac{E_b}{C_o} = 365.55\ \frac{lb}{ft}$

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

Restraint Panel Height = 10ft Maximum
Restraint Panel Width = 1ft-10-1/2 in Minimum
Allowable Shear per Panel = 1031 lbs Seismic & 1444 lbs Wind

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

Shear per Panel: $V_{s1} := (2\ ft \cdot E_b) = 731.10\ lb$ O.K.

$V_{s2} := (2 \cdot ft \cdot vb) = 336.05\ lb$ O.K.

Dead Load Resisting Overturning: $L_b := 2.83 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10 \cdot psf) \cdot 4 \cdot ft$ $DLRb := \frac{W_b \cdot L_b}{2} = 198.10\ lb$

Chord Force:

$CFb_w := \frac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b} = 1680.25\ lb$ $CFb_s := \frac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b} = 3655.48\ lb$

$CFb_w + CFbb_w = 2644.48\ lb$ $CFb_s + CFbb_s = 6070.69\ lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1561.39\ lb$ $HDFb_s := CFb_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRb = 3567.98\ lb$

Simpson STHD14/RJ

$HDFb_w + HDFbb_w = 2366.50\ lb$ $HDFb_s + HDFbb_s = 5866.05\ lb$

Simpson HDU8 w/ SB7/8x24 anchor

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vb} = 0.97\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.45\ ft$

16d Sinkers @ 4" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vb} = 8.19\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_b} = 3.76\ ft$

5/8" dia. anchors @ 32" o.c.

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PROJECT : **4024 85th AVE SE**

WALL C:

Story Shear due to Wind: $V_{2W} = 13600.00\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 31 \cdot ft$

Shear Wall Length: $Lc := \left(4.75\left(1.25 - 0.125\left(\frac{13}{4.75}\right)\right) + 10.25 + 8.17 + 15.33\right) \cdot ft = 38.06\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

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

$vc = 95.57\ \frac{lb}{ft}$ $\quad \frac{vc}{C_o} = 95.57\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $\quad E_c := \frac{E_{cc} \cdot Lcc + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Lc}$

$E_c = 126.03\ \frac{lb}{ft}$ $\quad \frac{E_c}{C_o} = 126.03\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_c := 4.75 \cdot ft$ Plate Height: $Pt := 13 \cdot ft$

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

$DLRc := \frac{W_c \cdot L_c}{2} = 332.50\ lb$

Chord Force:

$CFc_w := \frac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c} = 1242.39\ lb$

$CFc_w + CFcc_w = 1747.23\ lb$

$CFc_s := \frac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c} = 1638.37\ lb$

$CFc_s + CFcc_s = 2498.49\ lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 1042.89\ lb$

$HDFc_w + HDFcc_w = 1343.61\ lb$

$HDFc_s := CFc_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRc = 1491.50\ lb$

$HDFc_s + HDFcc_s = 2201.35\ lb$

Simpson STHD10/RJ

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vc} = 1.71\ ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 1.29\ ft$

16d Sinkers @ 12" o.c.

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

$Z_B := 860 \cdot lb$ $\quad C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vc} = 14.40\ ft$ $\quad \frac{(Z_B \cdot C_D \cdot C_o)}{E_c} = 10.92\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

WALL D:

Story Shear due to Wind: $V_{2W} = 13600.00\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 15 \cdot ft$

Shear Wall Length: $Ld := \left(20.17 + 7.83 + 4.5\left(1.25 - 0.125\left(\frac{10}{4.5}\right)\right)\right) \cdot ft = 32.38\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

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

$vd = 59.76\ \frac{lb}{ft}$ $\qquad \frac{vd}{C_o} = 59.76\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $\qquad E_d := \frac{E_{dd} \cdot Ldd + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Ld}$

$E_d = 80.88\ \frac{lb}{ft}$ $\qquad \frac{E_d}{C_o} = 80.88\ \frac{lb}{ft}$

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

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

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

$DLRd := \frac{W_d \cdot L_d}{2} = 247.50\ lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d} = 597.61\ lb$

$CFd_w + CFdd_w = 931.41\ lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d} = 808.85\ lb$

$CFd_s + CFdd_s = 1377.57\ lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6 \cdot DLRd = 449.11\ lb$

$HDFd_w + HDFdd_w = 656.91\ lb$

$HDFd_s := CFd_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRd = 699.52\ lb$

$HDFd_s + HDFdd_s = 1175.49\ lb$

Simpson LSTHD8/RJ

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vd} = 2.73\ ft$ $\qquad \frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 2.02\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $\qquad C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vd} = 23.03\ ft$ $\qquad \frac{(Z_B \cdot C_D \cdot C_o)}{E_d} = 17.01\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

WALL E:

Story Shear due to Wind: $V_{2W} = 13600.00\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 31 \cdot ft$ $L_2 := 23 \cdot ft$

Shear Wall Length: $Le := (6 + 13.5) \cdot ft = 19.50\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

Wind Force: $ve := \frac{vee \cdot Lee + \left(\frac{0.6 \cdot V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le}$

$ve = 315.99\ \frac{lb}{ft}$ $\frac{ve}{C_o} = 315.99\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_e := \frac{E_{ee} \cdot Lee + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le}$

$E_e = 413.25\ \frac{lb}{ft}$ $\frac{E_e}{C_o} = 413.25\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_e := 6 \cdot ft$

Plate Height: $Pt := 10 \cdot ft$

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

$DLRe := \frac{W_e \cdot L_e}{2} = 330.00\ lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e} = 3159.89\ lb$

$CFe_w + CFee_w = 3956.39\ lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e} = 4132.55\ lb$

$CFe_s + CFee_s = 5489.57\ lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 2961.89\ lb$

$HDFe_w + HDFee_w = 3614.39\ lb$

$HDFe_s := CFe_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRe = 3986.79\ lb$

$HDFe_s + HDFee_s = 5237.80\ lb$

Simpson HDU5 w/ SB5/8x24 anchor

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{ve} = 0.52\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 0.39\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{ve} = 4.35\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_e} = 3.33\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

WALL F:

Story Shear due to Wind: $V_{2W} = 13600.00\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 69 \cdot ft$

Distance between shear walls: $L_1 := 15 \cdot ft$ $L_2 := 23 \cdot ft$

Shear Wall Length: $Lf := (18) \cdot ft = 18.00\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

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

$vf = 247.72\ \frac{lb}{ft}$ $\frac{vf}{C_o} = 247.72\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_f := \frac{E_{ff} \cdot Lff + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lf}$

$E_f = 326.67\ \frac{lb}{ft}$ $\frac{E_f}{C_o} = 326.67\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_f := 18 \cdot ft$

Plate Height: $Pt := 10 \cdot ft$

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

$DLRf := \frac{W_f \cdot L_f}{2} = 990.00\ lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f} = 2477.20\ lb$

$CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f} = 3266.75\ lb$

$CFf_w + CFff_w = 3747.65\ lb$

$CFf_s + CFff_s = 5431.26\ lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 1883.20\ lb$

$HDFf_s := CFf_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRf = 2829.46\ lb$

Simpson HDU2 w/ SSTB16 anchor

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vf} = 0.66\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 0.50\ ft$

16d Sinkers @ 6" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vf} = 5.55\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_f} = 4.21\ ft$

5/8" dia. anchors @ 48" o.c.

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PROJECT : **4024 85th AVE SE**

WALL G:

Story Shear due to Wind: $V_{4W} = 8726.94\ lb$

Story Shear due to Seismic: $F_2 = 8426.42\ lb$

Bldg Width in direction of Load: $L_t := 44.5 \cdot ft$

Distance between shear walls: $L_1 := 21.5 \cdot ft$ $L_2 := 23 \cdot ft$

Shear Wall Length: $Lg := (20 + 11) \cdot ft = 31.00\ ft$

Percent full height sheathing: $\frac{10 \cdot ft}{10 \cdot ft} \cdot 100 = 100.00$

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

Wind Force: $vg := \frac{\left(\frac{0.6 \cdot V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg}$

$vg = 84.45\ \frac{lb}{ft}$ $\frac{vg}{C_o} = 84.45\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_g := \frac{\left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg}$

$E_g = 123.68\ \frac{lb}{ft}$ $\frac{E_g}{C_o} = 123.68\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_g := 11 \cdot ft$

Plate Height: $Pt := 10 \cdot ft$

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

$DLRg := \frac{W_g \cdot L_g}{2} = 1210.00\ lb$

Chord Force:

$CFg_w := \frac{vg \cdot L_g \cdot Pt}{C_o \cdot L_g} = 844.54\ lb$

$CFg_s := \frac{E_g \cdot L_g \cdot Pt}{C_o \cdot L_g} = 1236.78\ lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6 \cdot DLRg = 118.54\ lb$

$HDFg_s := CFg_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRg = 702.32\ lb$

No Holdowns Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vg} = 1.93\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_g} = 1.32\ ft$

16d Sinkers @ 16" o.c.

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

$Z_B := 860 \cdot lb$ $C_D := 1.6$

$\frac{(Z_B \cdot C_D \cdot C_o)}{vg} = 16.29\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_g} = 11.13\ ft$

5/8" dia. anchors @ 72" o.c.

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PROJECT : **4024 85th AVE SE**

Diaphragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa \cdot \frac{Laa}{69 \cdot ft} = 39.59 \ \frac{lb}{ft} \qquad E_{aa} \cdot \frac{Laa}{69 \cdot ft} = 99.18 \ \frac{lb}{ft}$$

Wall Lines BB:

$$vbb \cdot \frac{Lbb}{69 \cdot ft} = 39.59 \ \frac{lb}{ft} \qquad E_{bb} \cdot \frac{Lbb}{69 \cdot ft} = 99.18 \ \frac{lb}{ft}$$

Wall Lines CC:

$$vcc \cdot \frac{Lcc}{40 \cdot ft} = 45.11 \ \frac{lb}{ft} \qquad E_{cc} \cdot \frac{Lcc}{40 \cdot ft} = 76.86 \ \frac{lb}{ft}$$

Wall Lines DD:

$$vdd \cdot \frac{Ldd}{40 \cdot ft} = 26.19 \ \frac{lb}{ft} \qquad E_{dd} \cdot \frac{Ldd}{40 \cdot ft} = 44.63 \ \frac{lb}{ft}$$

Wall Lines EE:

$$vee \cdot \frac{Lee}{38 \cdot ft} = 78.12 \ \frac{lb}{ft} \qquad E_{ee} \cdot \frac{Lee}{38 \cdot ft} = 133.10 \ \frac{lb}{ft}$$

Wall Lines FF:

$$vff \cdot \frac{Lff}{38 \cdot ft} = 58.21 \ \frac{lb}{ft} \qquad E_{ff} \cdot \frac{Lff}{38 \cdot ft} = 99.18 \ \frac{lb}{ft}$$

Wall Lines A:

$$\frac{va \cdot La - vaa \cdot Laa}{50 \cdot ft} = 27.06 \ \frac{lb}{ft}$$

$$\frac{E_a \cdot La - E_{aa} \cdot Laa}{50 \cdot ft} = 39.63 \ \frac{lb}{ft}$$

Wall Lines B:

$$\frac{vb \cdot Lb - vbb \cdot Lbb}{69 \cdot ft} = 18.33 \ \frac{lb}{ft}$$

$$\frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{69 \cdot ft} = 26.85 \ \frac{lb}{ft}$$

Wall Lines C:

$$\frac{vc \cdot Lc - vcc \cdot Lcc}{44 \cdot ft} = 41.66 \ \frac{lb}{ft}$$

$$\frac{E_c \cdot Lc - E_{cc} \cdot Lcc}{44 \cdot ft} = 39.15 \ \frac{lb}{ft}$$

Wall Lines D:

$$\frac{vd \cdot Ld - vdd \cdot Ldd}{40 \cdot ft} = 22.17 \ \frac{lb}{ft}$$

$$\frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{40 \cdot ft} = 20.84 \ \frac{lb}{ft}$$

Wall Line E:

$$\frac{ve \cdot Le - vee \cdot Lee}{40 \cdot ft} = 79.83 \ \frac{lb}{ft}$$

$$\frac{E_e \cdot Le - E_{ee} \cdot Lee}{40 \cdot ft} = 75.01 \ \frac{lb}{ft}$$

Wall Line F:

$$\frac{vf \cdot Lf - vff \cdot Lff}{40 \cdot ft} = 56.17 \ \frac{lb}{ft}$$

$$\frac{E_f \cdot Lf - E_{ff} \cdot Lff}{40 \cdot ft} = 52.79 \ \frac{lb}{ft}$$

Wall Lines G:

$$vg \cdot \frac{Lg}{69 \cdot ft} = 37.94 \ \frac{lb}{ft}$$

$$E_g \cdot \frac{Lg}{69 \cdot ft} = 55.57 \ \frac{lb}{ft}$$

Mark Myers, PE

04/18/2025

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Cantilevered Retaining Wall

Project File: 40245 85th AVE SE Beams.ec6

LIC# : KW-06015659, Build:20.25.04.16 MYERS ENGINEERING (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 4ft Stem at Basement w/ Slab

Code Reference

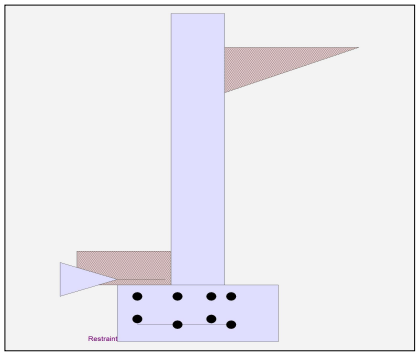
Calculations per IBC 2018, ACI 318-14, TMS 402-16

Criteria

Retained Height	=	3.50 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	6.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	125.00 pcf
Soil Density, Toe	=	125.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	150.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	1.3 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		Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Cantilevered Retaining Wall

Project File: 40245 85th AVE SE Beams.ec6

DESCRIPTION: 4ft Stem at Basement w/ Slab

Design Summary

Wall Stability Ratios

Overturning	=	2.77	OK
Slab Resists All Sliding !			
Global Stability	=	2.64	
Total Bearing Load	=	1,298 lbs	
...resultant ecc.	=	1.18 in	
Eccentricity within middle third			
Soil Pressure @ Toe	=	840 psf	OK
Soil Pressure @ Heel	=	458 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,027 psf	
ACI Factored @ Heel	=	559 psf	
Footing Shear @ Toe	=	1.2 psi	OK
Footing Shear @ Heel	=	9.6 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	328.6 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Bottom		
		Stem OK		
Design Height Above Ftg	ft =	0.00		
Wall Material Above "Ht"	=	Concrete		
Design Method	=	SD	SD	SD
Thickness	=	8.00		
Rebar Size	=	# 4		
Rebar Spacing	=	10.00		
Rebar Placed at	=	6 in		
Design Data				
fb/FB + fa/Fa	=	**0.061**		
Total Force @ Section				
Service Level	lbs =			
Strength Level	lbs =	343.0		
Moment....Actual				
Service Level	ft-# =			
Strength Level	ft-# =	381.4		
Moment.....Allowable	=	6,174.1		
Shear.....Actual				
Service Level	psi =			
Strength Level	psi =	4.8		
Shear.....Allowable	psi =	75.0		
Anet (Masonry)	in2 =			
Wall Weight	psf =	100.0		
Rebar Depth 'd'	in =	6.00		

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	2,500.0
Fy	psi =	60,000.0

Cantilevered Retaining Wall

DESCRIPTION: 4ft Stem at Basement w/ Slab

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0149 in2/ft		
(4/3) * As :	0.0199 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(6)/60000 :	0.24 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.24 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.8128 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	0.67 ft
Heel Width	=	1.33
Total Footing Width	=	2.00
Footing Thickness	=	10.00 in

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 7.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,027	559	psf
Mu' : Upward	=	217	136	ft-#
Mu' : Downward	=	50	326	ft-#
Mu: Design	=	167	190	ft-#
φ Mn	=	6,985	2,665	ft-#
Actual 1-Way Shear	=	1.22	9.63	psi
Allow 1-Way Shear	=	75.00	75.00	psi

Toe Reinforcing = # 4 @ 10.00 in
Heel Reinforcing = # 4 @ 10.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, φ Tn = 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@ 18 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: #4@ 11.11 in, #5@ 17.22 in, #6@ 18 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Key: No key defined

Min footing T&S reinf Area 0.43 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

DESCRIPTION: 4ft Stem at Basement w/ Slab

Summary of Overturning & Resisting Forces & Moments

Item	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#	.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	328.6	1.44	474.7			
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 =						
=						
				Soil Over HL (ab. water tbl)	291.5	1.67	485.8
				Soil Over HL (bel. water tbl)		1.67	485.8
				Water Table			
				Sloped Soil Over Heel =			
				Surcharge Over Heel =			
				Adjacent Footing Load =			
				Axial Dead Load on Stem =	150.0	1.10	165.6
				* Axial Live Load on Stem =			
				Soil Over Toe =	41.7	0.33	13.9
				Surcharge Over Toe =			
				Stem Weight(s) =	400.0	1.00	400.0
				Earth @ Stem Transitions =			
				Footing Weight =	250.0	1.00	249.9
				Key Weight =			
				Vert. Component =			
Total =	328.6	**O.T.M.** =	474.7	**Total =**	1,133.2 lbs	**R.M.=**	1,315.3

Resisting/Overturning Ratio = **2.77**

Vertical Loads used for Soil Pressure = 1,298.0 lbs

* 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 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)	0.047	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

DESCRIPTION: 4ft Stem at Basement w/ Slab

Rebar Lap & Embedment Lengths Information

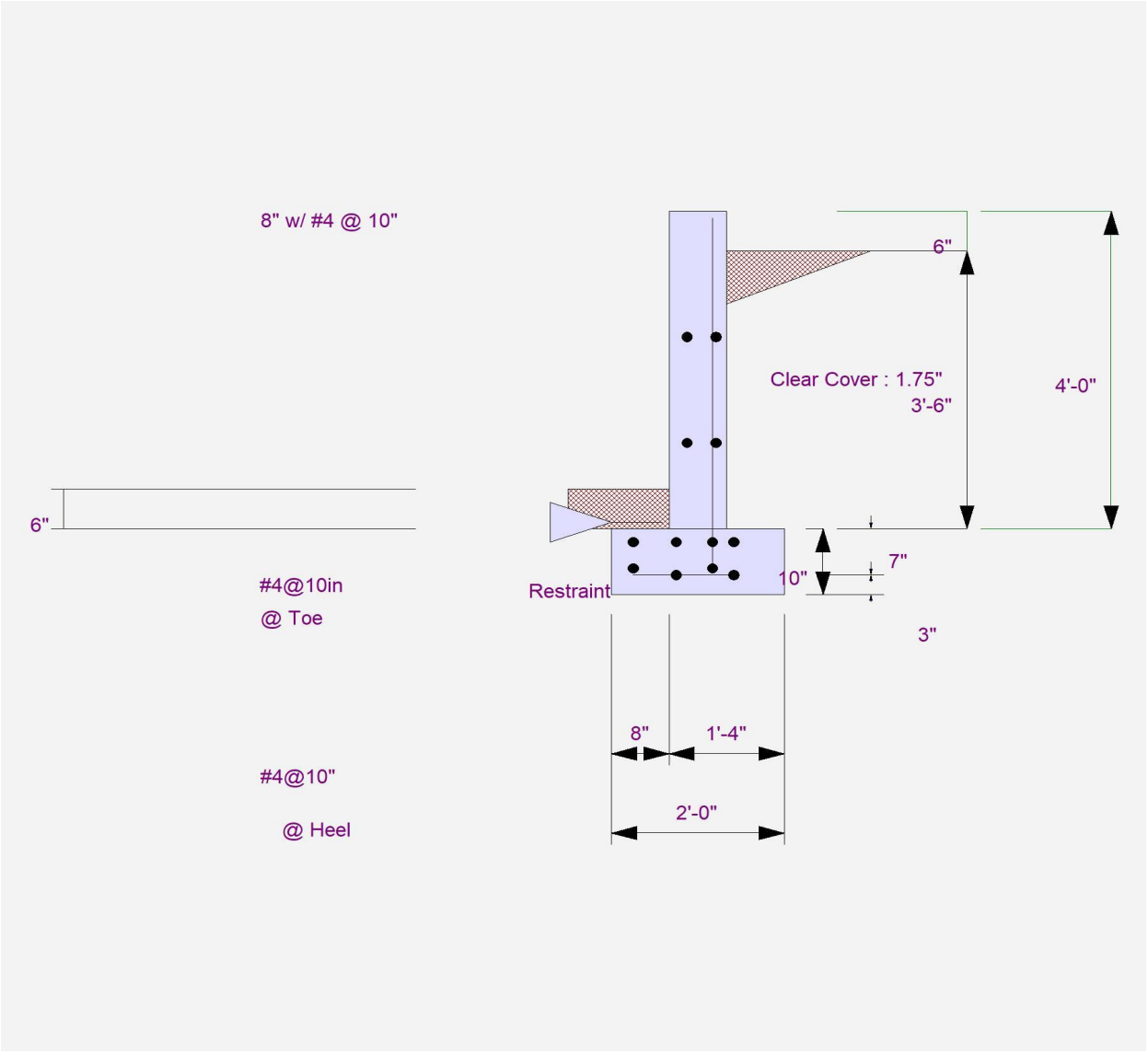
Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.3a) =	18.72 in
Development length for #4 bar specified in this stem design segment =	14.40 in
Hooked embedment length into footing for #4 bar specified in this stem design segment =	6.05 in
As Provided =	0.2400 in2/ft
As Required =	0.1728 in2/ft

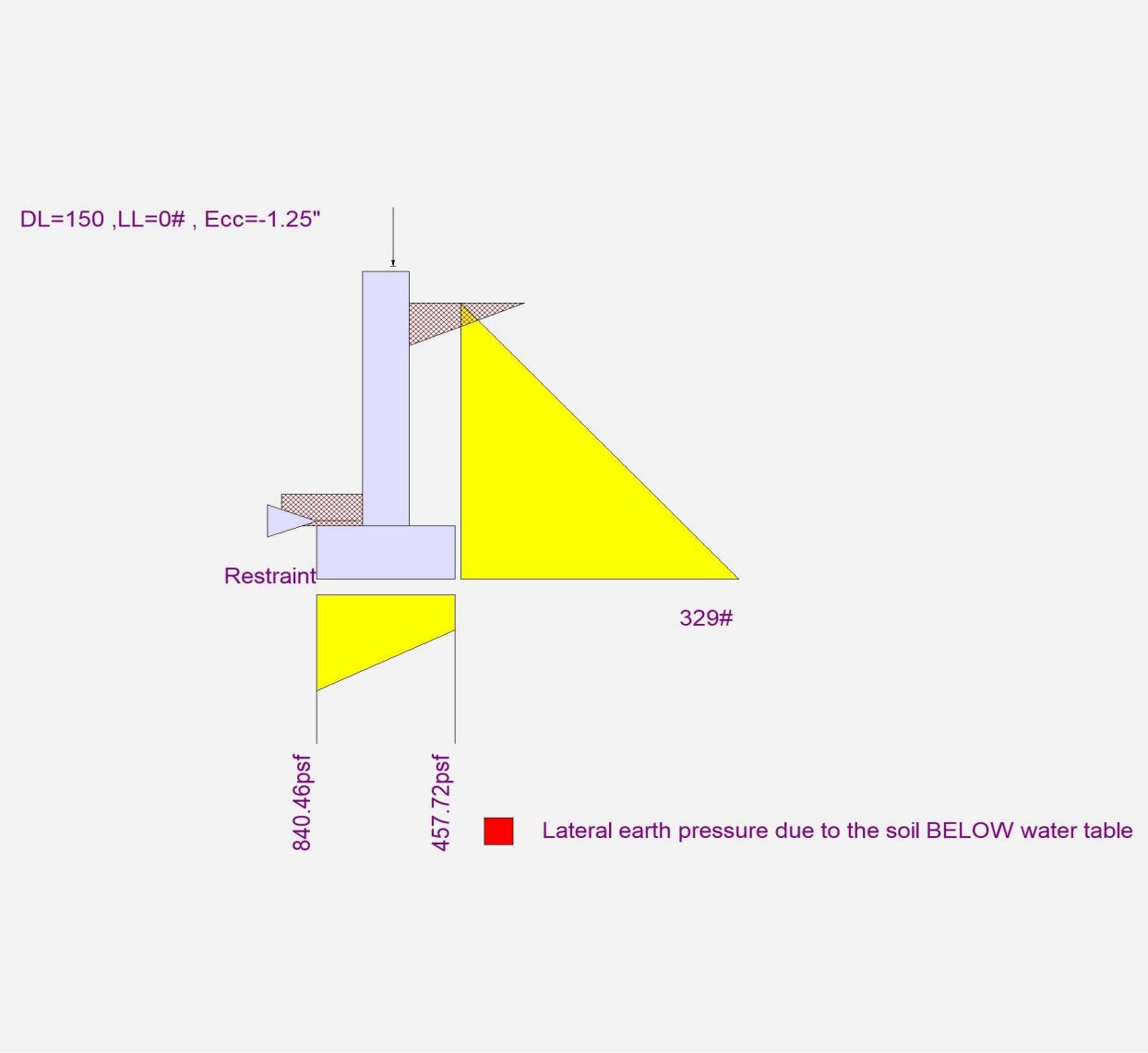
Cantilevered Retaining Wall

DESCRIPTION: 4ft Stem at Basement w/ Slab



Cantilevered Retaining Wall

DESCRIPTION: 4ft Stem at Basement w/ Slab



Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Cantilevered Retaining Wall

Project File: 40245 85th AVE SE Beams.ec6

DESCRIPTION: 6ft Stem at Basement w/ Slab

Code Reference

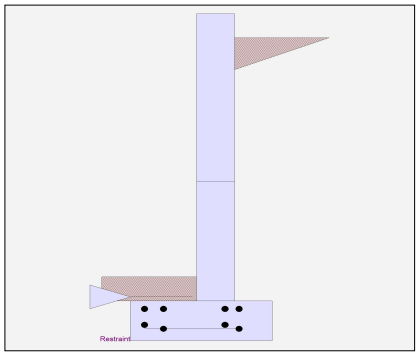
Calculations per IBC 2018, ACI 318-14, TMS 402-16

Criteria

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

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	125.00 pcf
Soil Density, Toe	=	125.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	150.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	1.3 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		Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Cantilevered Retaining Wall

Project File: 40245 85th AVE SE Beams.ec6

DESCRIPTION: 6ft Stem at Basement w/ Slab

Design Summary

Wall Stability Ratios

Overturning	=	1.73	OK
Slab Resists All Sliding !			
Global Stability	=	1.84	
Total Bearing Load	=	1,946 lbs	
...resultant ecc.	=	2.88 in	
Eccentricity within middle third			
Soil Pressure @ Toe	=	1,227 psf	OK
Soil Pressure @ Heel	=	330 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,406 psf	
ACI Factored @ Heel	=	378 psf	
Footing Shear @ Toe	=	8.2 psi	OK
Footing Shear @ Heel	=	29.0 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	701.9 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		2nd	Bottom		
		Stem OK	Stem OK		
Design Height Above Ftg	ft =	2.50	0.00		
Wall Material Above "Ht"	=	Concrete	Concrete		
Design Method	=	SD	SD	SD	SD
Thickness	=	8.00	8.00		
Rebar Size	=	# 4	# 4		
Rebar Spacing	=	10.00	10.00		
Rebar Placed at	=	6 in	6 in		
Design Data					
fb/FB + fa/Fa	=	**0.037**	**0.248**		
Total Force @ Section					
Service Level	lbs =				
Strength Level	lbs =	252.0	847.0		
Moment....Actual					
Service Level	ft-# =				
Strength Level	ft-# =	233.3	1,534.1		
Moment.....Allowable	ft-# =	6,174.1	6,174.1		
Shear.....Actual					
Service Level	psi =				
Strength Level	psi =	3.5	11.8		
Shear.....Allowable	psi =	75.0	75.0		
Anet (Masonry)	in2 =				
Wall Weight	psf =	100.0	100.0		
Rebar Depth 'd'	in =	6.00	6.00		

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Cantilevered Retaining Wall

DESCRIPTION: 6ft Stem at Basement w/ Slab

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0091 in2/ft		
(4/3) * As :	0.0122 in2/ft	Min Stem T&S Reinf Area 0.672 in2	
200bd/fy : 200(12)(6)/60000 :	0.24 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.24 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.8128 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.06 in2/ft		
(4/3) * As :	0.08 in2/ft	Min Stem T&S Reinf Area 0.480 in2	
200bd/fy : 200(12)(6)/60000 :	0.24 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.24 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.8128 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.17 ft
Heel Width	=	1.33
Total Footing Width	=	2.50
Footing Thickness	=	10.00 in

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 7.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,406	378	psf
Mu' : Upward	=	849	104	ft-#
Mu' : Downward	=	153	592	ft-#
Mu: Design	=	696	488	ft-#
φ Mn	=	6,985	2,665	ft-#
Actual 1-Way Shear	=	8.24	28.99	psi
Allow 1-Way Shear	=	75.00	75.00	psi

Toe Reinforcing = # 4 @ 10.00 in
Heel Reinforcing = # 4 @ 10.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, φ Tn = 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@ 18 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: #4@ 11.11 in, #5@ 17.22 in, #6@ 18 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Key: No key defined

Min footing T&S reinf Area 0.54 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

DESCRIPTION: 6ft Stem at Basement w/ Slab

Summary of Overturning & Resisting Forces & Moments

Item	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#	.....RESISTING..... 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 =						
=						
Soil Over HL (ab. water tbl)				458.1	2.17	992.6
Soil Over HL (bel. water tbl)					2.17	992.6
Water Table						
Sloped Soil Over Heel =						
Surcharge Over Heel =						
Adjacent Footing Load =						
Axial Dead Load on Stem =				150.0	1.60	240.7
* Axial Live Load on Stem =						
Soil Over Toe =				72.9	0.58	42.6
Surcharge Over Toe =						
Stem Weight(s) =				600.0	1.50	900.2
Earth @ Stem Transitions =						
Footing Weight =				312.5	1.25	390.6
Key Weight =						
Vert. Component =						
Total =	701.9	**O.T.M.** =	1,481.9	**Total =** 1,593.5 lbs	**R.M.=**	2,566.7

Resisting/Overturning Ratio = **1.73**

Vertical Loads used for Soil Pressure = 1,945.6 lbs

* 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)	0.082	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

DESCRIPTION: 6ft Stem at Basement w/ Slab

Rebar Lap & Embedment Lengths Information

Stem Design Segment: 2nd
Stem Design Height: 2.50 ft above top of footing

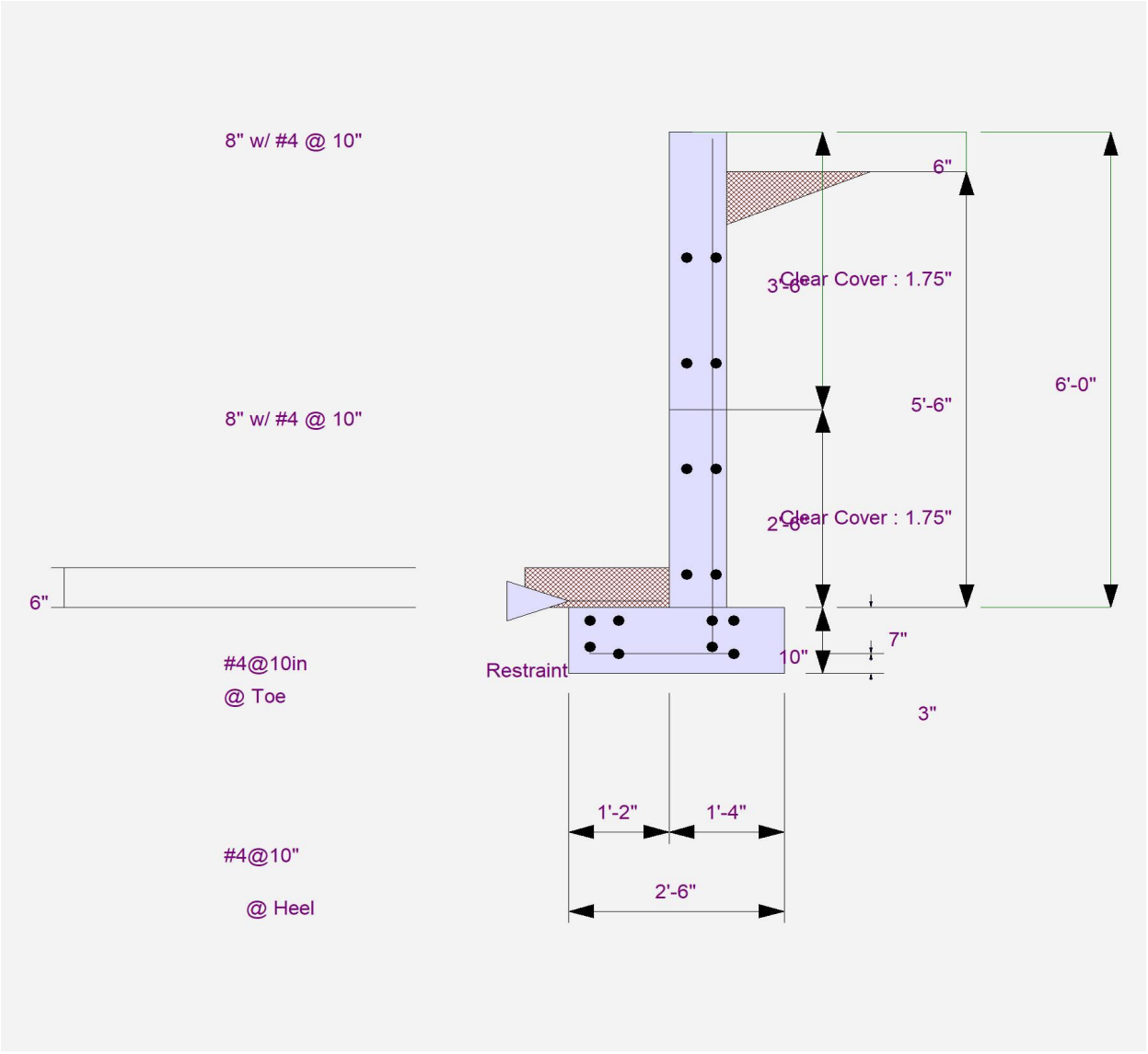
Lap Splice length for #4 bar specified in this stem design segment (25.4.2.3a) =	18.72 in
Development length for #4 bar specified in this stem design segment =	14.40 in

Stem Design Segment: Bottom
Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.3a) =	18.72 in
Development length for #4 bar specified in this stem design segment =	14.40 in
Hooked embedment length into footing for #4 bar specified in this stem design segment =	6.05 in
As Provided =	0.2400 in2/ft
As Required =	0.1728 in2/ft

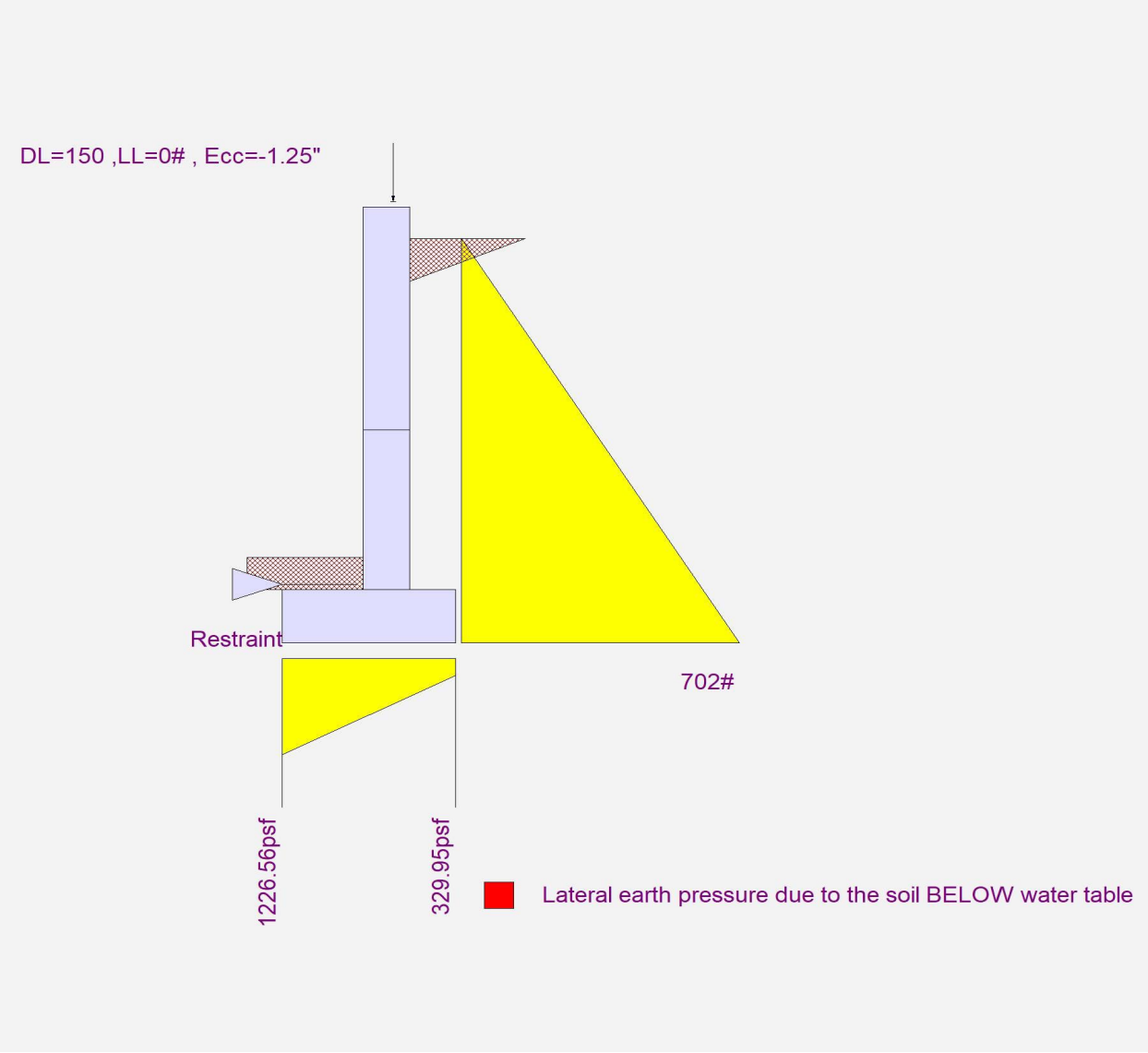
Cantilevered Retaining Wall

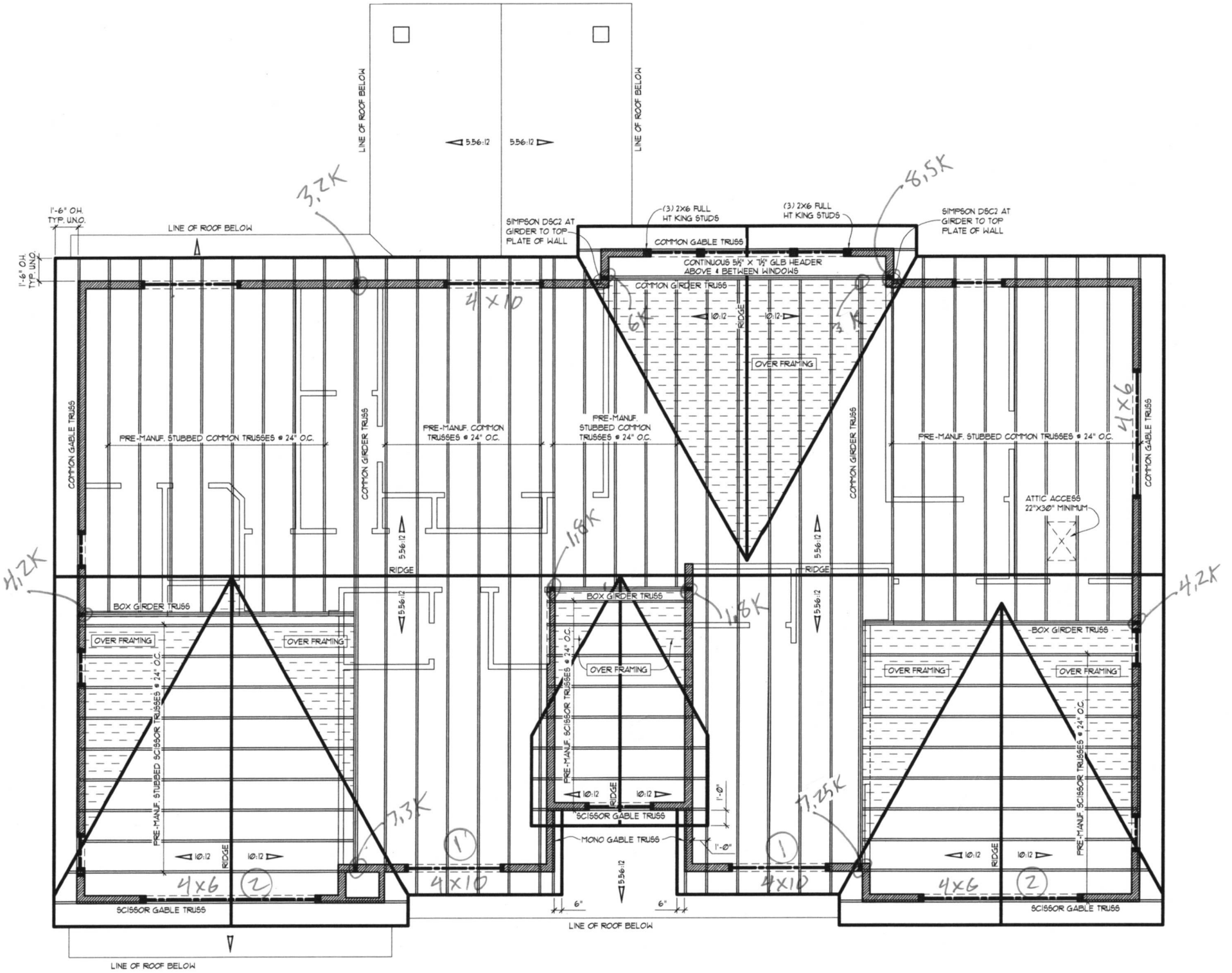
DESCRIPTION: 6ft Stem at Basement w/ Slab

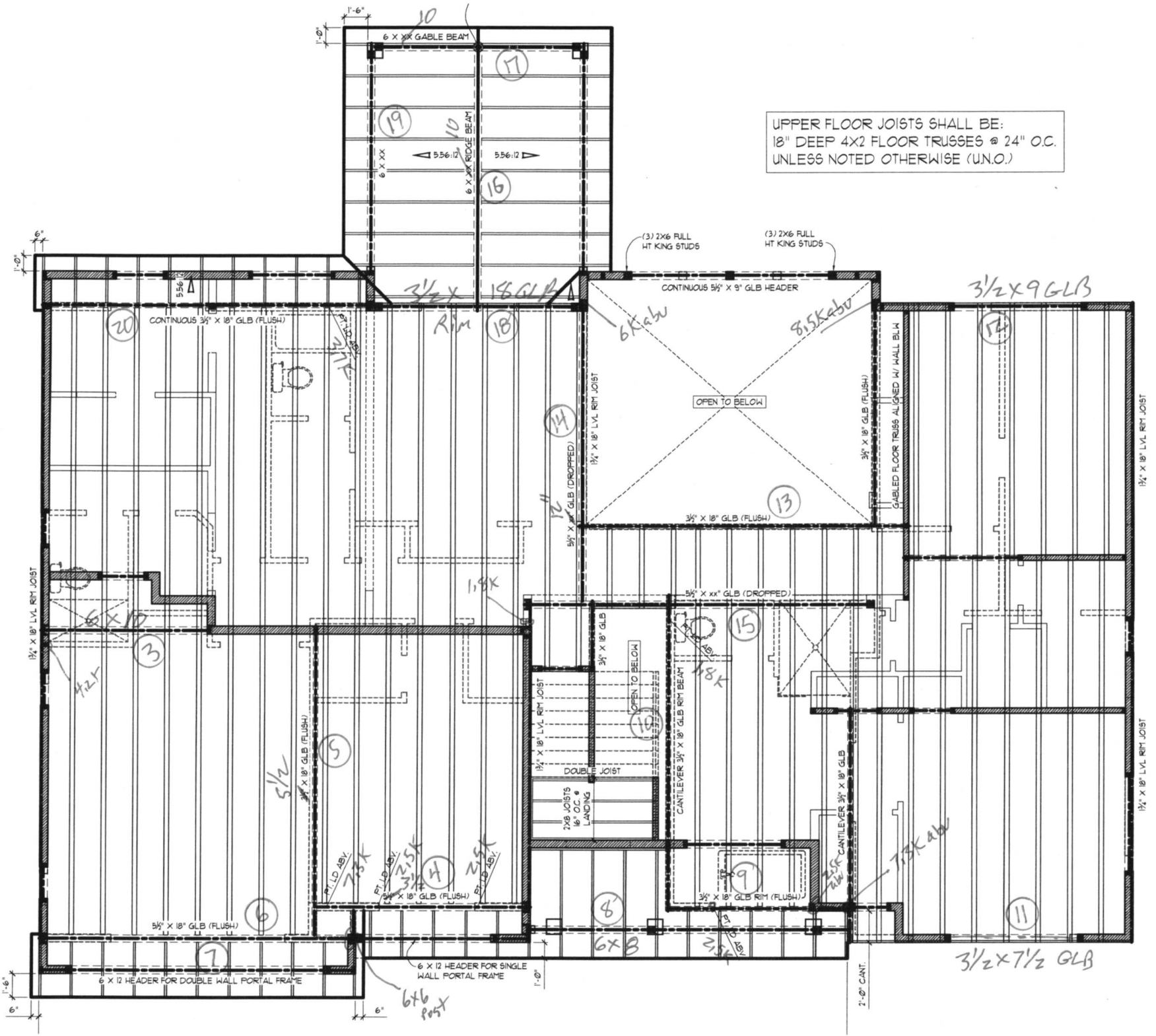


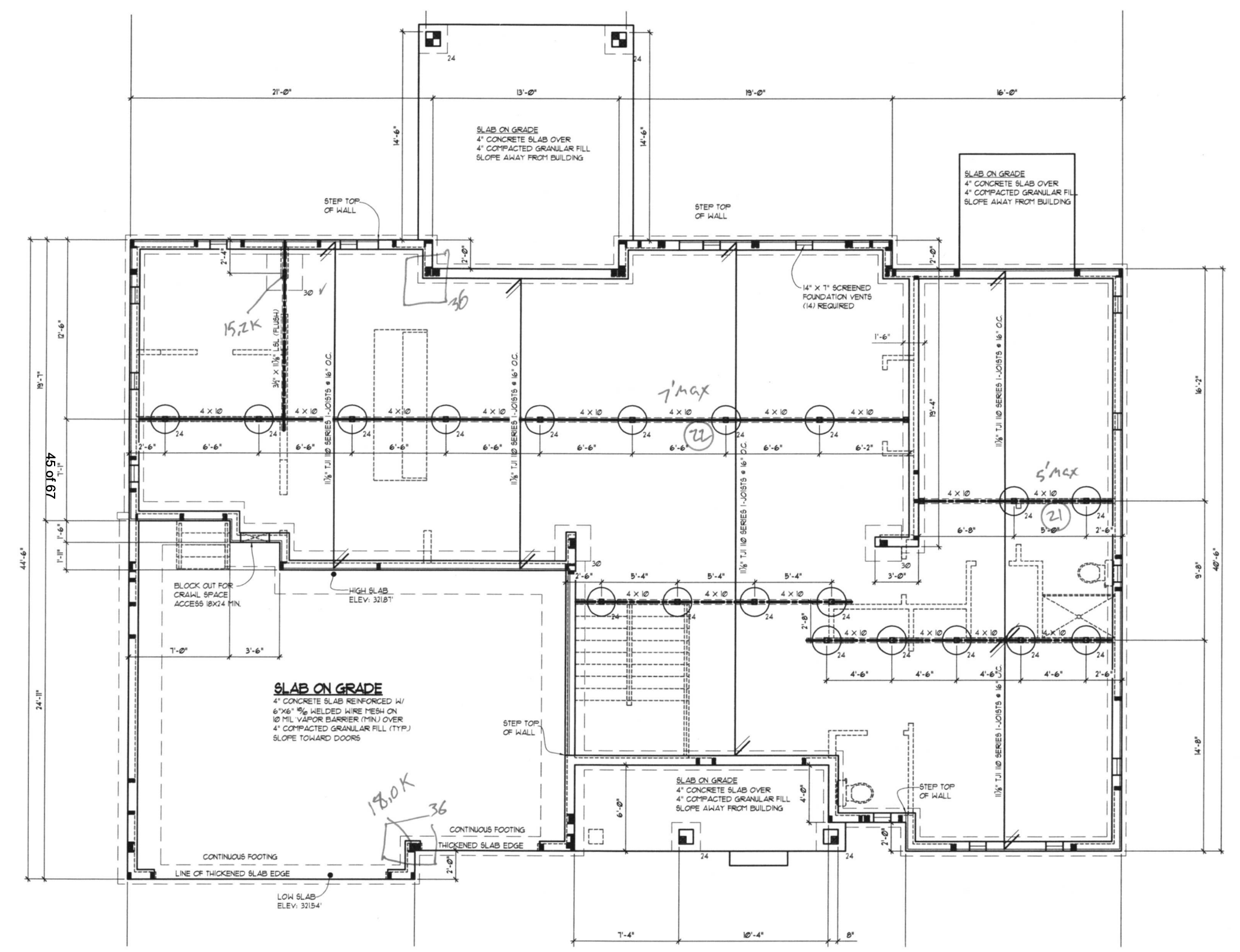
Cantilevered Retaining Wall

DESCRIPTION: 6ft Stem at Basement w/ Slab




LINE OF ROOF BELOW
5.56:12
5.56:12
LINE OF ROOF BELOW
3.2K
1'-6" O.H. TYP. U.N.O.
LINE OF ROOF BELOW
SIMPSON DSC2 AT GIRDER TO TOP PLATE OF WALL
(3) 2X6 FULL HT KING STUDS
(3) 2X6 FULL HT KING STUDS
6.5K
SIMPSON DSC2 AT GIRDER TO TOP PLATE OF WALL
COMMON GABLE TRUSS
CONTINUOUS 5½" X 11⅞" GLB HEADER ABOVE & BETWEEN WINDOWS
COMMON GIRDER TRUSS
1'-6" O.H. TYP. U.N.O.
4X10
6K
10:12
RIDGE
10:12
3 K
OVER FRAMING
4X6
COMMON GABLE TRUSS
PRE-MANUF. STUBBED COMMON TRUSSES @ 24" O.C.
COMMON GIRDER TRUSS
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
PRE-MANUF. STUBBED COMMON TRUSSES @ 24" O.C.
COMMON GIRDER TRUSS
PRE-MANUF. STUBBED COMMON TRUSSES @ 24" O.C.
COMMON GABLE TRUSS
ATTIC ACCESS 22"X30" MINIMUM
4.2K
5.56:12
RIDGE
1.8K
5.56:12
RIDGE
4.2K
BOX GIRDER TRUSS
OVER FRAMING
OVER FRAMING
PRE-MANUF. STUBBED SCISSOR TRUSSES @ 24" O.C.
10:12
RIDGE
10:12
7.3K
4X6
2
1
4X10
5.56:12
BOX GIRDER TRUSS
OVER FRAMING
PRE-MANUF. SCISSOR TRUSSES @ 24" O.C.
10:12
RIDGE
10:12
SCISSOR GABLE TRUSS
MONO GABLE TRUSS
1'-0"
1'-0"
1
4X10
7.25K
5.56:12
6"
6"
BOX GIRDER TRUSS
OVER FRAMING
OVER FRAMING
PRE-MANUF. SCISSOR TRUSSES @ 24" O.C.
10:12
RIDGE
10:12
4X6
2
SCISSOR GABLE TRUSS
SCISSOR GABLE TRUSS
LINE OF ROOF BELOW
LINE OF ROOF BELOW


UPPER FLOOR JOISTS SHALL BE:
18" DEEP 4X2 FLOOR TRUSSES @ 24" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)
6 X XX GABLE BEAM
6 X XX RIDGE BEAM
5.56:12
(3) 2X6 FULL HT KING STUDS
CONTINUOUS 5½" X 9" GLB HEADER
CONTINUOUS 3½" X 18" GLB (FLUSH)
OPEN TO BELOW
GABLED FLOOR TRUSS ALIGNED W/ WALL BLW
5½" X xx" GLB (DROPPED)
1¾" X 18" LVL RIM JOIST
3½" X 18" GLB (FLUSH)
3½" X 18" GLB
3½" X 18" GLB RIM BEAM
CANTILEVER 3½" X 18" GLB
DOUBLE JOIST
2X8 JOISTS 16" O.C. @ LANDING
3½" X 18" GLB RIM (FLUSH)
FT LD ABV.
6 X 12 HEADER FOR SINGLE WALL PORTAL FRAME
6 X 12 HEADER FOR DOUBLE WALL PORTAL FRAME
6x6 Post
6x8
3½x9 GLB
3½x7½ GLB
2'-0" CANT.


SLAB ON GRADE
4" CONCRETE SLAB OVER
4" COMPACTED GRANULAR FILL
SLOPE AWAY FROM BUILDING
STEP TOP OF WALL
14" X 7" SCREENED FOUNDATION VENTS (14) REQUIRED
11⅞" TJI 110 SERIES I-JOISTS @ 16" O.C.
4 X 10
BLOCK OUT FOR CRAWL SPACE ACCESS 18X24 MIN.
HIGH SLAB ELEV: 321.87'
LOW SLAB ELEV: 321.54'
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/ 6"X6" 10/6 WELDED WIRE MESH ON 10 MIL VAPOR BARRIER (MIN.) OVER 4" COMPACTED GRANULAR FILL (TYP.) SLOPE TOWARD DOORS
CONTINUOUS FOOTING
THICKENED SLAB EDGE
LINE OF THICKENED SLAB EDGE
15.2K
18.0K
7'max
5'max

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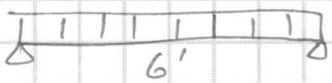
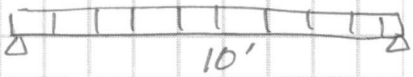
① $w_D = 15psf\left(4'/_2\right) = 315plf$
$w_S = 25psf\left(4'/_2\right) = 525plf$



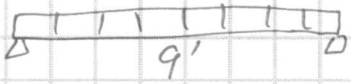
4x10

② $w_D = 15psf(3') = 45plf$
$w_S = 25psf(3') = 75plf$



4x6

③ $w_D = 15psf\left(23'/_2\right) = 172.5plf$
$w_L = 40psf\left(23'/_2\right) = 460plf$



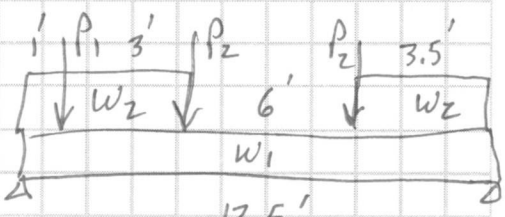
6x10 DF#1

④ $w_{D1} = 15psf\left(20'/_2\right) + 12psf(9') = 258plf$
$w_{L1} = 40psf\left(20'/_2\right) = 400plf$

$w_{D2} = 15psf\left(4'/_2\right) = 315plf$
$w_{S2} = 25psf\left(4'/_2\right) = 525plf$

$P_1 = 2955^{\#}DL + 4345^{\#}SL$ from Girder

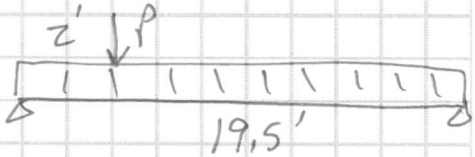
$P_2 = 945^{\#}DL + 1555^{\#}SL$ from Header



~~3½~~ x18 GLB
5½ for Hanger

⑤ $w_D = 15plf$
$w_L = 40plf$

$P = 6600^{\#}DL + 2700^{\#}LL + 7550^{\#}SL$ from ④



5½x18 GLB

FOR NWEB
JOB 4024 85th SE

DATE 4-17-25
BY MM

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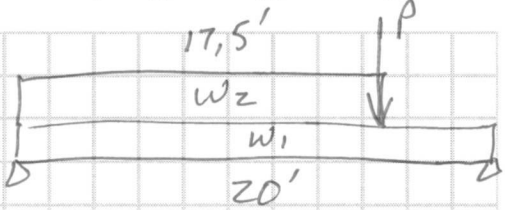
(6) $w_{D1} = 15psf(3' + 5'/2) + 12psf(9') = 190.5plf$
$w_{S1} = 25psf(3' + 5'/2) = 137.5plf$

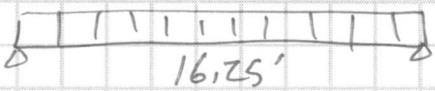
$w_{D2} = 15psf(20'/2) = 150plf$
$w_{L2} = 40psf(20'/2) = 400plf$

$P = 6070\#DL + 2810\#LL + 6780\#SL$ from (5)



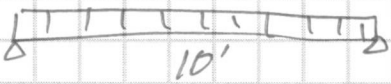
5½ x 18 GLB

(7) $w_D = 15psf(5'/2) = 37.5plf$
$w_S = 25psf(5'/2) = 62.5plf$



6 x 12

(8) $w_D = 15psf(9'/2) = 67.5plf$
$w_S = 25psf(9'/2) = 112.5plf$

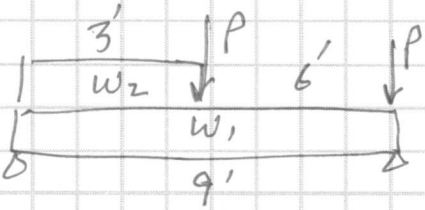


6 x 8

(9) $w_{D1} = 15psf(5'/2) + 12psf(9') = 145.5plf$
$w_{L1} = 40psf(4'/2) = 80plf$
$w_{S1} = 25psf(1') = 25plf$

$w_{D2} = 315plf$
$w_{S2} = 525plf$

$P = 945\#DL + 1555\#SL$ from header



3½ x 18 GLB

FOR NWEB
JOB 4024 85th SE

DATE 4-17-25
BY MM

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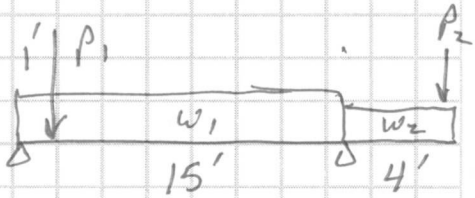
myengineer@centurytel.net

(10) $w_{D1} = 15plf + 12psf(13') + 15psf(12'/2) = 261plf$
$w_{L1} = 40plf$
$w_{S1} = 25psf(12'/2) = 150plf$

$w_{D2} = 15psf(2') + 12psf(9') = 138plf$
$w_{L2} = 40plf$
$w_{S2} = 25psf(2') = 50plf$

$P_1 = 730\#DL + 1070\#SL$ from Box Girder

$P_2 = 2090\#DL + 360\#LL + 2490\#SL$ from (9)



3½x18 GLB

(11) $w_D = 12psf(9') + 15psf(15'/2 + 3') = 265.5plf$
$w_L = 40psf(15'/2) = 300plf$
$w_S = 25psf(3') = 75plf$

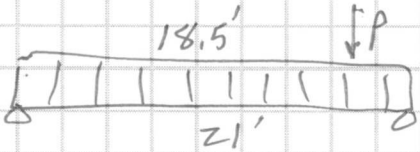
4x12 OR
3½x7½ GLB

(12) $w_D = 15psf(25'/2 + 16'/2) + 12psf(9) = 415.5plf$
$w_L = 40psf(16'/2) = 320plf$
$w_S = 25psf(25'/2) = 312.5plf$

3½x9 GLB

(13) $w_D = 15psf(5'/2) = 37.5plf$
$w_L = 40psf(5'/2) = 100plf$

$P = 105\#DL + 280\#LL \pm 2200\#EL \pm 1300\#WL$ from Beam



3½x18 GLB

FOR NWEB
JOB 4024 85TH SE

DATE 4-17-25
BY MH

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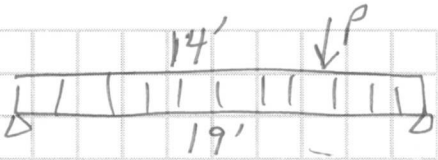
(14) $w_D = 15plf + 12psf(9') = 123plf$
$w_L = 40plf$

$P_1 = 410^{\#}DL + 1080^{\#}LL \pm 150^{\#}WL \pm 260^{\#}EL$ from (13)

$P_2 = \pm 800^{\#}WL \pm 1400^{\#}EL$ from wall EE



5½ x 12 GLB

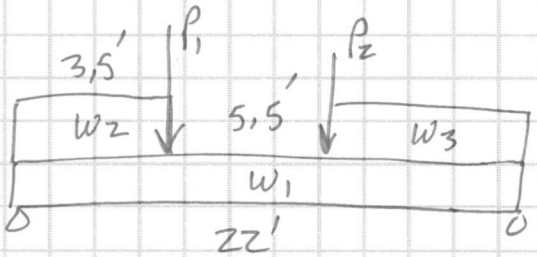
(15) $w_{D1} = 15psf(5'/2) = 37.5plf$
$w_{L1} = 40psf(5'/2) = 100plf$

$w_{D2} = 15psf(19'/2) = 142.5plf$
$w_{L2} = 40psf(19'/2) = 380plf$

$w_{D1} = 15psf(15'/2) = 112.5plf$
$w_{L2} = 40psf(15'/2) = 300plf$

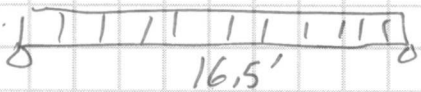
$P_1 = 1470^{\#}DL + 1180^{\#}LL \pm 700^{\#}WL \pm 1220^{\#}EL$ from (14)

$P_2 = 2030^{\#}DL + 300^{\#}LL + 1460^{\#}SL$ from (10)



6¾ x 18 GLB

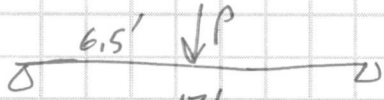
(16) $w_D = 15psf(14'/2) = 105plf$
$w_S = 25psf(14'/2) = 175plf$



6 x 10 DF#1

(17) $w_D = self$

$P = 867^{\#}DL + 1444^{\#}SL$ from (16)



6 x 10 DF#1

FOR NWEB
JOB 4024 85th SE

DATE 4-17-25
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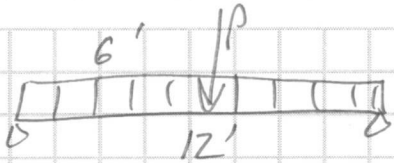
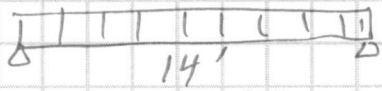
(18) $w_D = 15psf\left(\frac{42'}{2}+\frac{21'}{2}\right)+12psf(9') = 580.5plf$
$w_L = 40psf\left(\frac{21'}{2}\right) = 420plf$
$w_S = 25psf\left(\frac{42'}{2}\right) = 525plf$

$P = 667\#DL + 1444\#SL$ from (16)



3½ x 16½ GLB
Minimum

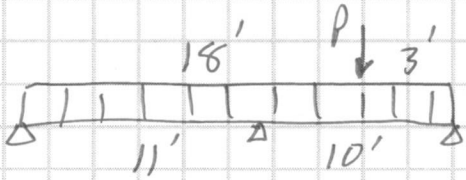
(19) $w_D = 15psf\left(\frac{10'}{2}\right) = 75plf$
$w_S = 25psf\left(\frac{10'}{2}\right) = 125plf$



6x6 DF #1
Minimum

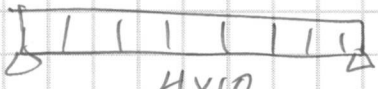
(20) $w_D = 15psf\left(\frac{25'}{2}+\frac{23'}{2}\right)+12psf(9') = 468plf$
$w_L = 40psf\left(\frac{21'}{2}\right) = 420plf$
$w_S = 25psf\left(\frac{25'}{2}+2'\right) = 362.5plf$

$P = 1500\#DL + 2200\#SL$ from Girder



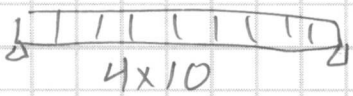
3½ x 18 GLB

(21) $w_D = 15psf\left(\frac{26'}{2}+\frac{26'}{2}\right) = 390plf$
$w_L = 40psf\left(\frac{26'}{2}+\frac{26'}{2}\right) = 1040plf$



4x10
5' max span

(22) $w_D = 15psf\left(\frac{25'}{2}\right) = 187.5plf$
$w_L = 40psf\left(\frac{25'}{2}\right) = 500plf$



4x10
7' max span

FOR NWEB
JOB 4024 85th SE

DATE 4-17-25
BY

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Multiple Simple Beam

Project File: 40245 85th AVE SE Beams.ec6

LIC# : KW-06015659, Build:20.25.04.16 MYERS ENGINEERING (c) ENERCALC, LLC 1982-2025

Description :

Wood Beam Design : 1. 6ft Header Upper Floor

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

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

Applied Loads

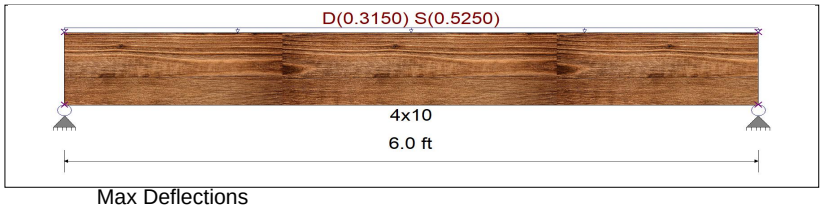
Unif Load: D = 0.3150, S = 0.5250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.740** : 1
fb : Actual : 908.81 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,227.30 psi
Load Comb : +D+S

Max fv/FvRatio = **0.421** : 1
fv : Actual : 87.18 psi at 5.240 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.95			1.58			
Right Support	0.95			1.58			



Max Deflections

Transient Downward	0.042 in	Total Downward	0.067 in
Ratio	1727	Ratio	1079
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 2. Gable wall header

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

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

Applied Loads

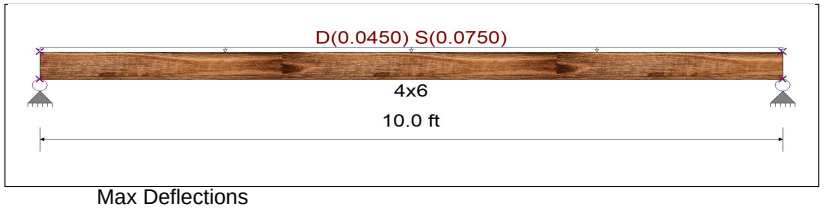
Unif Load: D = 0.0450, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.767** : 1
fb : Actual : 1,020.07 psi at 5.000 ft in Span # 1
Fb : Allowable : 1,329.96 psi
Load Comb : +D+S

Max fv/FvRatio = **0.206** : 1
fv : Actual : 42.70 psi at 9.567 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.23			0.38			
Right Support	0.23			0.38			



Max Deflections

Transient Downward	0.219 in	Total Downward	0.350 in
Ratio	549	Ratio	343
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : 3. Dropped beam at rear of Garage

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch				Wood Grade :	No.1			
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

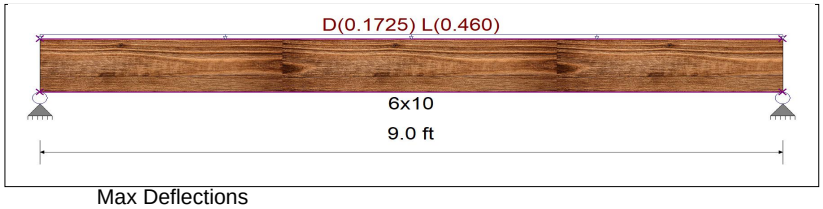
Unif Load: D = 0.1725, L = 0.460 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.688** : 1
fb : Actual : 928.92 psi at 4.500 ft in Span # 1
Fb : Allowable : 1,350.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.397** : 1
fv : Actual : 67.55 psi at 0.000 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.78		2.07				
Right Support	0.78		2.07				



Max Deflections

Transient Downward	0.109 in	Total Downward	0.149 in
Ratio	994	Ratio	723
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 4. Rim beam over single car

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF				Wood Grade :	24F-V4			
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

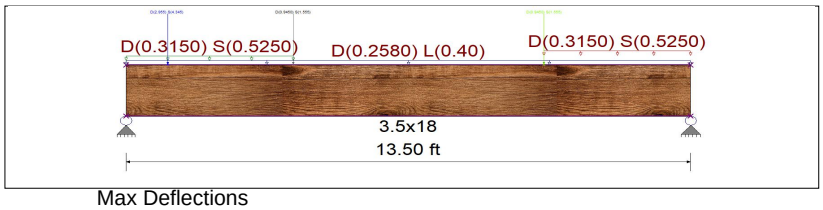
Unif Load: D = 0.2580, L = 0.40 k/ft, Trib= 1.0 ft
Unif Load: D = 0.3150, S = 0.5250 k/ft, 0.0 to 4.0 ft, Trib= 1.0 ft
Unif Load: D = 0.3150, S = 0.5250 k/ft, 10.0 to 13.50 ft, Trib= 1.0 ft
1Point: D = 2.955, S = 4.345 k @ 1.0 ft
2Point: D = 0.9450, S = 1.555 k @ 4.0 ft
3Point: D = 0.9450, S = 1.555 k @ 10.0 ft

Design Summary

Max fb/Fb Ratio = **0.670** : 1
fb : Actual : 1,847.93 psi at 5.625 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.556** : 1
fv : Actual : 169.38 psi at 12.015 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	6.60		2.70	7.55			
Right Support	4.09		2.70	3.84			



Max Deflections

Transient Downward	0.141 in	Total Downward	0.330 in
Ratio	1148	Ratio	490
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : 5. Transfer beam at Garage

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **5.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF				Wood Grade :	24F-V4			
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

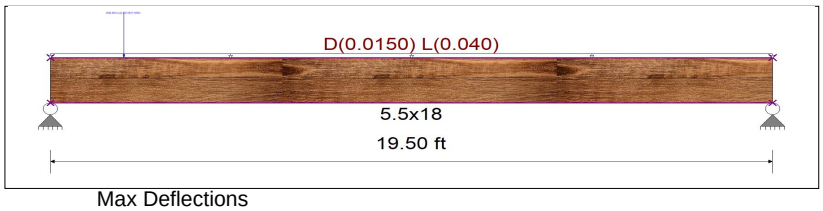
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.0 ft
1Point: D = 6.60, L = 2.70, S = 7.550 k @ 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.403** : 1
fb : Actual : 1,067.27 psi at 2.015 ft in Span # 1
Fb : Allowable : 2,651.26 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.656** : 1
fv : Actual : 199.90 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	6.07		2.81	6.78			
Right Support	0.82		0.67	0.77			



Max Deflections

Transient Downward	0.131 in	Total Downward	0.278 in
Ratio	1783	Ratio	841
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 6. Rim beam over 2 Car Garage

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **5.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF				Wood Grade :	24F-V4			
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

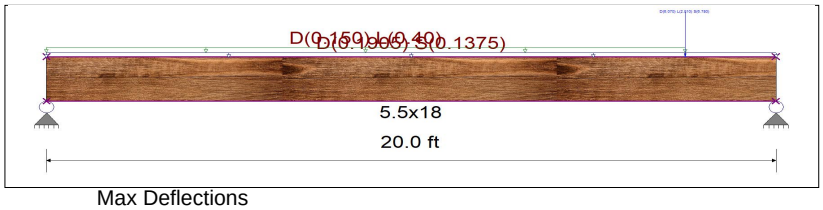
Unif Load: D = 0.1905, S = 0.1375 k/ft, Trib= 1.0 ft
Unif Load: D = 0.150, L = 0.40 k/ft, 0.0 to 17.50 ft, Trib= 1.0 ft
1Point: D = 6.070, L = 2.810, S = 6.780 k @ 17.50 ft

Design Summary

Max fb/Fb Ratio = **0.843** : 1
fb : Actual : 1,938.33 psi at 11.400 ft in Span # 1
Fb : Allowable : 2,299.61 psi
Load Comb : +D+L

Max fv/FvRatio = **0.873** : 1
fv : Actual : 265.99 psi at 18.533 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	4.14		4.29	2.22			
Right Support	8.36		5.52	7.31			



Max Deflections

Transient Downward	0.352 in	Total Downward	0.842 in
Ratio	681	Ratio	285
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Multiple Simple Beam

Project File: 40245 85th AVE SE Beams.ec6

Wood Beam Design : 7. Header at Garage Door

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.1		
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

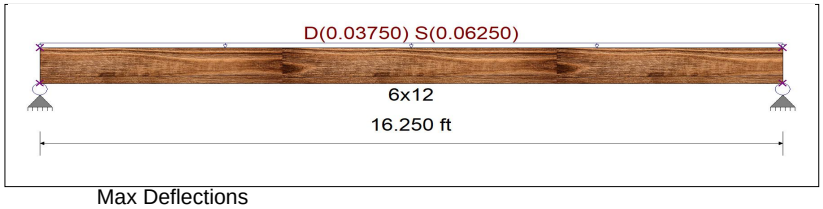
Unif Load: D = 0.03750, S = 0.06250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.215** : 1
fb : Actual : 326.73 psi at 8.125 ft in Span # 1
Fb : Allowable : 1,520.24 psi
Load Comb : +D+S

Max fv/FvRatio = **0.087** : 1
fv : Actual : 17.08 psi at 15.329 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.30			0.51			
Right Support	0.30			0.51			



Max Deflections

Transient Downward	0.088 in	Total Downward	0.141 in
Ratio	2206	Ratio	1378
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 8. Entry Porch Roof beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.1		
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

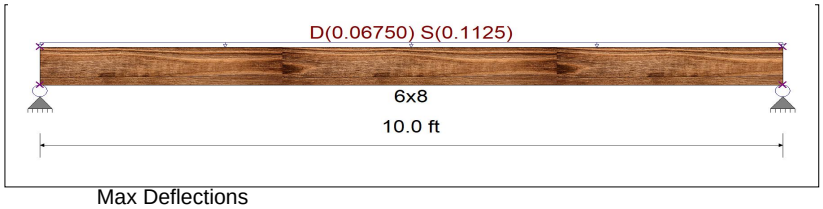
Unif Load: D = 0.06750, S = 0.1125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.340** : 1
fb : Actual : 523.64 psi at 5.000 ft in Span # 1
Fb : Allowable : 1,541.86 psi
Load Comb : +D+S

Max fv/FvRatio = **0.147** : 1
fv : Actual : 28.80 psi at 9.400 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.34			0.56			
Right Support	0.34			0.56			



Max Deflections

Transient Downward	0.082 in	Total Downward	0.132 in
Ratio	1458	Ratio	911
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : 9. Rim Beam at Master Bath

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

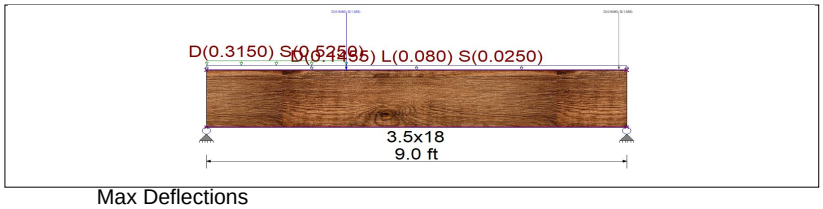
Unif Load: D = 0.1455, L = 0.080, S = 0.0250 k/ft, Trib= 1.0 ft
Unif Load: D = 0.3150, S = 0.5250 k/ft, 0.0 to 3.0 ft, Trib= 1.0 ft
1Point: D = 0.9450, S = 1.555 k @ 3.0 ft
2Point: D = 0.9450, S = 1.555 k @ 8.830 ft

Design Summary

Max fb/Fb Ratio = **0.212** : 1
fb : Actual : 583.88 psi at 3.000 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.239** : 1
fv : Actual : 72.99 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.09		0.36	2.49			
Right Support	2.05		0.36	2.42			



Max Deflections

Transient Downward	0.020 in	Total Downward	0.038 in
Ratio	5439	Ratio	2831
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 10. Cantilever Rim Beam at Master Bath

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

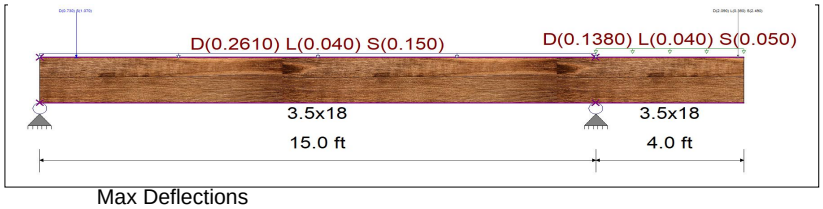
Unif Load: D = 0.2610, L = 0.040, S = 0.150 k/ft, 0.0 ft to 15.0 ft, Trib= 1.0 ft
Unif Load: D = 0.1380, L = 0.040, S = 0.050 k/ft, 15.0 to 19.0 ft, Trib= 1.0 ft
1Point: D = 0.730, S = 1.070 k @ 1.0 ft
2Point: D = 2.090, L = 0.360, S = 2.490 k @ 18.833 ft

Design Summary

Max fb/Fb Ratio = **0.569** : 1
fb : Actual : 1,210.10 psi at 15.000 ft in Span # 1
Fb : Allowable : 2,127.50 psi
Load Comb : +D+S+H

Max fv/FvRatio = **0.395** : 1
fv : Actual : 120.24 psi at 15.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S+H

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	2.03		0.30	1.46			
Right Support	5.26		0.93	4.55			



Max Deflections

Transient Downward	0.088 in	Total Downward	0.129 in
Ratio	1096	Ratio	740
	LC: S Only		LC: +D+S+H
Transient Upward	-0.024 in	Total Upward	-0.020 in
Ratio	7500	Ratio	8870
	LC: S Only		+0.750L+0.750S+H, LL C

Multiple Simple Beam

Description :

Wood Beam Design : 11. 8ft header Main floor

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

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

Applied Loads

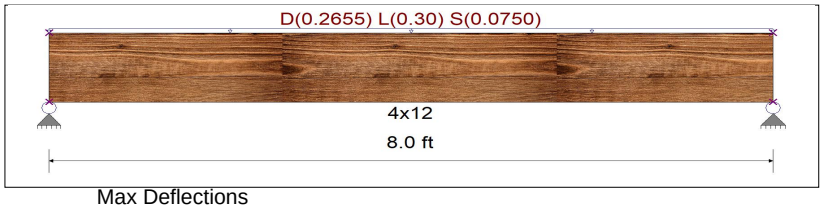
Unif Load: D = 0.2655, L = 0.30, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.755** : 1
fb : Actual : 735.33 psi at 4.000 ft in Span # 1
Fb : Allowable : 974.23 psi
Load Comb : +D+L

Max fv/FvRatio = **0.367** : 1
fv : Actual : 66.06 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.06		1.20	0.30			
Right Support	1.06		1.20	0.30			



Max Deflections

Transient Downward	0.042 in	Total Downward	0.079 in
Ratio	2294	Ratio	1217
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 11. 8ft header Main floor

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x7.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2400 psi	Fc - Prll	1650 psi	Fv	265 psi	Ebend- xx	1800 ksi	Density	31.21 pcf
Fb - Compr	1850 psi	Fc - Perp	650 psi	Ft	1100 psi	Eminbend - xx	950 ksi		

Applied Loads

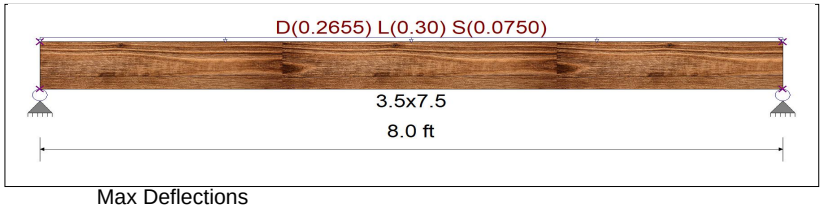
Unif Load: D = 0.2655, L = 0.30, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.697** : 1
fb : Actual : 1,654.49 psi at 4.000 ft in Span # 1
Fb : Allowable : 2,374.65 psi
Load Comb : +D+L

Max fv/FvRatio = **0.413** : 1
fv : Actual : 109.44 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.06		1.20	0.30			
Right Support	1.06		1.20	0.30			



Max Deflections

Transient Downward	0.125 in	Total Downward	0.237 in
Ratio	764	Ratio	405
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : 12. 8ft header at SGD

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2400 psi	Fc - Prll	1650 psi	Fv	265 psi	Ebend- xx	1800 ksi	Density	31.21 pcf
Fb - Compr	1850 psi	Fc - Perp	650 psi	Ft	1100 psi	Eminbend - xx	950 ksi		

Applied Loads

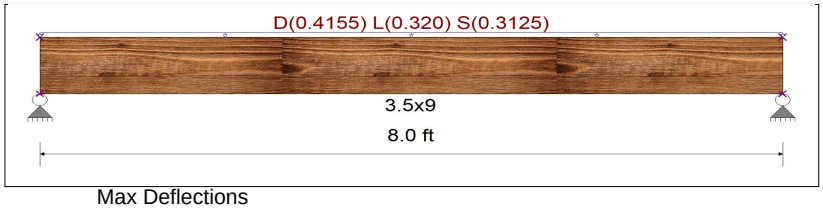
Unif Load: D = 0.4155, L = 0.320, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.666** : 1
fb : Actual : 1,808.00 psi at 4.000 ft in Span # 1
Fb : Allowable : 2,715.35 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.452** : 1
fv : Actual : 137.86 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.66		1.28	1.25			
Right Support	1.66		1.28	1.25			



Max Deflections

Transient Downward	0.077 in	Total Downward	0.215 in
Ratio	1239	Ratio	445
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 13. Walkway Rim beam over Great Rm

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

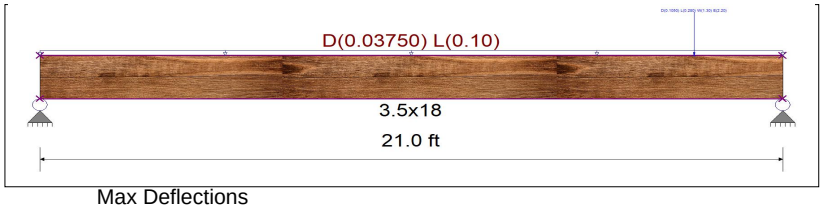
Unif Load: D = 0.03750, L = 0.10 k/ft, Trib= 1.0 ft
1Point: D = 0.1050, L = 0.280, W = 1.30, E = 2.20 k @ 18.50 ft

Design Summary

Max fb/Fb Ratio = **0.214** : 1
fb : Actual : 512.29 psi at 10.850 ft in Span # 1
Fb : Allowable : 2,394.22 psi
Load Comb : +D+L

Max fv/FvRatio = **0.257** : 1
fv : Actual : 108.79 psi at 19.530 ft in Span # 1
Fv : Allowable : 424.00 psi
Load Comb : +1.158D+2.10E

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.41		1.08		0.15	0.26	
Right Support	0.49		1.30		1.15	1.94	



Max Deflections

Transient Downward	0.154 in	Total Downward	0.218 in
Ratio	1631	Ratio	1154
	LC: L Only		LC: +D+0.750L+0.5250E
Transient Upward	-0.086 in	Total Upward	-0.027 in
Ratio	2919	Ratio	9185
	LC: E Only * -1.0		LC: +0.60D-0.70E

Multiple Simple Beam

Wood Beam Design : 14. Supporting dropped beam over Dining/Great Rm

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **5.5x12, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF					Wood Grade :	24F-V4		
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

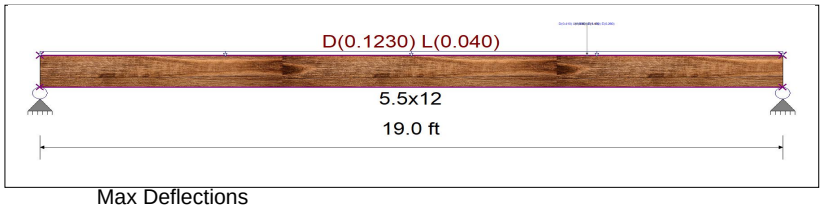
Applied Loads

Unif Load: D = 0.1230, L = 0.040 k/ft, Trib= 1.0 ft
1Point: D = 0.410, L = 1.080, W = 0.150, E = 0.260 k @ 14.0 ft
2Point: W = 0.80, E = 1.40 k @ 14.0 ft

Design Summary

Max fb/Fb Ratio = **0.478** : 1
fb : Actual : 1,833.80 psi at 13.997 ft in Span # 1
Fb : Allowable : 3,840.00 psi
Load Comb : +1.119D+0.750L+1.575E

Max fv/FvRatio = **0.230** : 1
fv : Actual : 97.60 psi at 18.050 ft in Span # 1
Fv : Allowable : 424.00 psi
Load Comb : +1.119D+0.750L+1.575E



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.28		0.66		0.25	0.44	
Right Support	1.47		1.18		0.70	1.22	

Max Deflections

Transient Downward	0.219 in	Total Downward	0.579 in
Ratio	1042	Ratio	393
	LC: L Only		LC: +D+0.750L+0.5250E
Transient Upward	-0.210 in	Total Upward	0.000 in
Ratio	1084	Ratio	9999
	LC: E Only * -1.0		LC:

Wood Beam Design : 15. Dropped beam over Foyer/Great Rm

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6.75x18, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF					Wood Grade :	24F-V4		
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

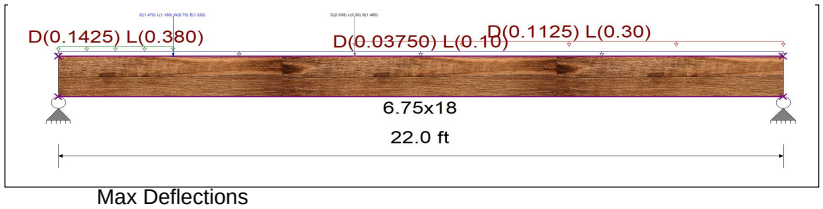
Applied Loads

Unif Load: D = 0.03750, L = 0.10 k/ft, Trib= 1.0 ft
Unif Load: D = 0.1425, L = 0.380 k/ft, 0.0 to 3.50 ft, Trib= 1.0 ft
Unif Load: D = 0.1125, L = 0.30 k/ft, 9.0 to 22.0 ft, Trib= 1.0 ft
1Point: D = 1.470, L = 1.180, W = 0.70, E = 1.220 k @ 3.50 ft
2Point: D = 2.030, L = 0.30, S = 1.460 k @ 9.0 ft

Design Summary

Max fb/Fb Ratio = **0.622** : 1
fb : Actual : 1,388.34 psi at 9.607 ft in Span # 1
Fb : Allowable : 2,231.62 psi
Load Comb : +D+L

Max fv/FvRatio = **0.346** : 1
fv : Actual : 91.57 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.74		4.65	0.86	0.59	1.03	
Right Support	2.55		4.26	0.60	0.11	0.19	

Max Deflections

Transient Downward	0.337 in	Total Downward	0.618 in
Ratio	782	Ratio	427
	LC: L Only		+0.750L+0.750S+0.5250E
Transient Upward	-0.038 in	Total Upward	0.000 in
Ratio	7022	Ratio	9999
	LC: E Only * -1.0		LC:

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Multiple Simple Beam

Project File: 40245 85th AVE SE Beams.ec6

LIC# : KW-06015659, Build:20.25.04.16 MYERS ENGINEERING (c) ENERCALC, LLC 1982-2025

Wood Beam Design : 16. Covered Patio Ridge

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.1

Fb - Tension	1350 psi	Fc - Prll	925 psi	Fv	170 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	1350 psi	Fc - Perp	625 psi	Ft	675 psi	Eminbend - xx	580 ksi		

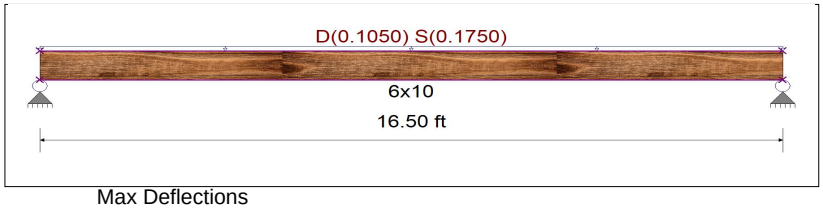
Applied Loads

Unif Load: D = 0.1050, S = 0.1750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.890** : 1
fb : Actual : 1,382.16 psi at 8.250 ft in Span # 1
Fb : Allowable : 1,552.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.308** : 1
fv : Actual : 60.13 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.87			1.44			
Right Support	0.87			1.44			

Max Deflections

Transient Downward	0.467 in	Total Downward	0.747 in
Ratio	424	Ratio	265
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 17. Covered Patio Gable Beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.1

Fb - Tension	1350 psi	Fc - Prll	925 psi	Fv	170 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	1350 psi	Fc - Perp	625 psi	Ft	675 psi	Eminbend - xx	580 ksi		

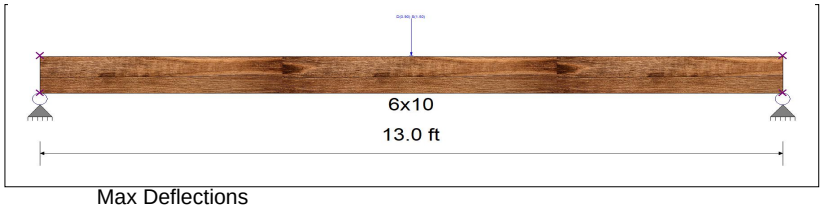
Applied Loads

Beam self weight calculated and added to loads
1Point: D = 0.90, S = 1.50 k @ 6.50 ft

Design Summary

Max fb/Fb Ratio = **0.760** : 1
fb : Actual : 1,166.10 psi at 6.500 ft in Span # 1
Fb : Allowable : 1,533.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.186** : 1
fv : Actual : 36.31 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.52			0.75			
Right Support	0.52			0.75			

Max Deflections

Transient Downward	0.190 in	Total Downward	0.315 in
Ratio	822	Ratio	495
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : 18. Header at Covered Patio SGD

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **3.5x16.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	DF/DF				Wood Grade :	24F-V4			
Fb - Tension	2400 psi	Fc - Prll	1650 psi	Fv	265 psi	Ebend- xx	1800 ksi	Density	31.21 pcf
Fb - Compr	1850 psi	Fc - Perp	650 psi	Ft	1100 psi	Eminbend - xx	950 ksi		

Applied Loads

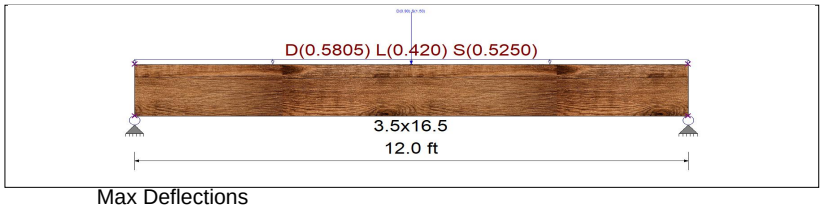
Beam self weight calculated and added to loads
Unif Load: D = 0.5805, L = 0.420, S = 0.5250 k/ft, Trib= 1.0 ft
1Point: D = 0.90, S = 1.50 k @ 6.0 ft

Design Summary

Max fb/Fb Ratio = **0.884** : 1
fb : Actual : 2,229.56 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,522.64 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.601** : 1
fv : Actual : 183.19 psi at 10.640 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	4.01		2.52	3.90			
Right Support	4.01		2.52	3.90			



Max Deflections

Transient Downward	0.144 in	Total Downward	0.313 in
Ratio	998	Ratio	460
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 19. Cov'd Patio side beam

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **6x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch				Wood Grade :	No.1			
Fb - Tension	1350 psi	Fc - Prll	925 psi	Fv	170 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	1350 psi	Fc - Perp	625 psi	Ft	675 psi	Eminbend - xx	580 ksi		

Applied Loads

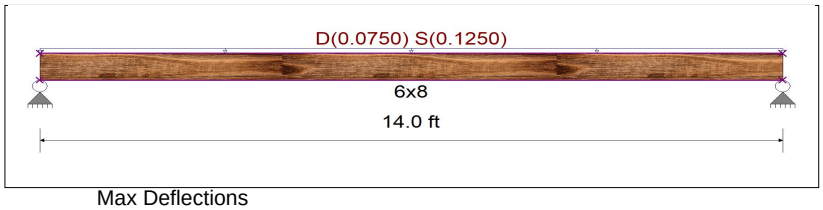
Unif Load: D = 0.0750, S = 0.1250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.735** : 1
fb : Actual : 1,140.36 psi at 7.000 ft in Span # 1
Fb : Allowable : 1,552.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.238** : 1
fv : Actual : 46.50 psi at 13.393 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.53			0.88			
Right Support	0.53			0.88			



Max Deflections

Transient Downward	0.351 in	Total Downward	0.562 in
Ratio	478	Ratio	299
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Project Title: 4024 85th SE
Engineer: Mark Myers, PE
Project ID:
Project Descr: Single Family Residence

Wood Beam

Project File: 40245 85th AVE SE Beams.ec6

DESCRIPTION: 20. Rim Beam over Kitchen/Prep

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

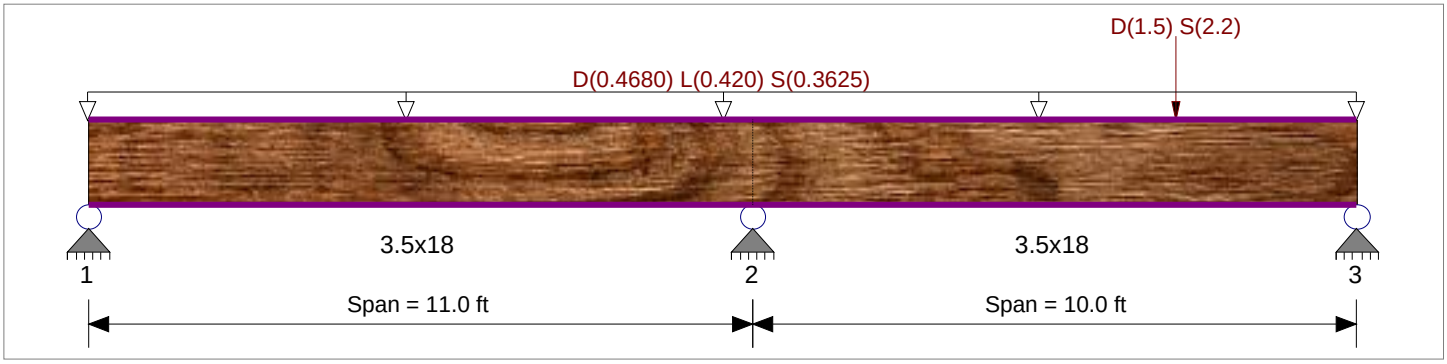
Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2400 psi
Fb -	1850 psi
Fc - Prll	1650 psi
Fc - Perp	650 psi
Fv	265 psi
Ft	1100 psi

E : Modulus of Elasticity

Ebend- xx	1800 ksi
Eminbend - xx	950 ksi
Density	31.21 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.4680, L = 0.420, S = 0.3625 k/ft
Load for Span Number 2
Point Load : D = 1.50, S = 2.20 k @ 7.0 ft, (Girder Truss)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.498 : 1	Maximum Shear Stress Ratio	=	0.497 : 1
Section used for this span		3.5x18	Section used for this span		3.5x18
fb: Actual	=	1,059.29 psi	fv: Actual	=	151.32 psi
F'b	=	2,127.50 psi	F'v	=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	11.000 ft	Location of maximum on span	=	11.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.026 in	Ratio = 4701	>=480	Span: 2 : S Only
Max Upward Transient Deflection	0 in	Ratio = 0	<480	n/a
Max Downward Total Deflection	0.051 in	Ratio = 2375	>=360	Span: 2 : +D+0.750L+0.750S
Max Upward Total Deflection	-0.000 in	Ratio = 758281	>=360	Span: 1 : +D+S

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+0.750L+0.750S	0.0448	4.609		0.0000	0.000
2	+D+0.750L+0.750S	0.0505	5.922		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	4.285	15.206	5.811
Max Upward from Load Combinations	4.285	15.206	5.811
Max Upward from Load Cases	1.895	6.790	2.707

Wood Beam

DESCRIPTION: 20. Rim Beam over Kitchen/Prep

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
D Only	1.895	6.790	2.643
+D+L	3.675	12.312	4.160
+D+S	3.302	12.489	5.350
+D+0.750L	3.230	10.932	3.781
+D+0.750L+0.750S	4.285	15.206	5.811
+0.60D	1.137	4.074	1.586
L Only	1.780	5.523	1.517
S Only	1.407	5.699	2.707

Multiple Simple Beam

Description :

Wood Beam Design : 21. Crawl Beam at bearing wall

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

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

Applied Loads

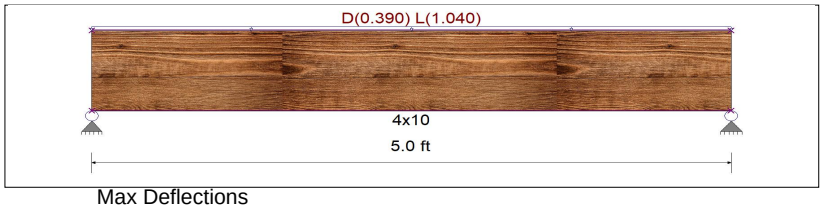
Unif Load: D = 0.390, L = 1.040 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.995** : 1
fb : Actual : 1,074.40 psi at 2.500 ft in Span # 1
Fb : Allowable : 1,080.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.638** : 1
fv : Actual : 114.84 psi at 4.233 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.98		2.60				
Right Support	0.98		2.60				



Max Deflections

Transient Downward	0.040 in	Total Downward	0.055 in
Ratio	1507	Ratio	1096
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 22. Crawl Beam NOT at bearing wall

Calculations per NDS 2018, IBC 2018, CBC 2019

BEAM Size : **4x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

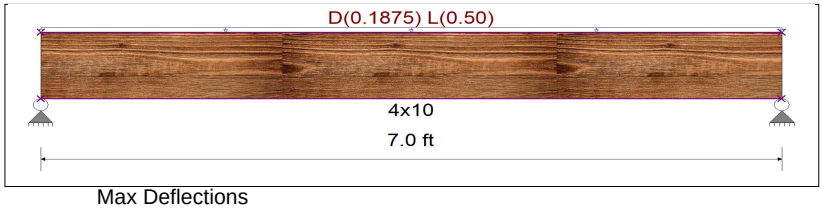
Unif Load: D = 0.1875, L = 0.50 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.937** : 1
fb : Actual : 1,012.42 psi at 3.500 ft in Span # 1
Fb : Allowable : 1,080.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.483** : 1
fv : Actual : 86.96 psi at 6.230 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.66		1.75				
Right Support	0.66		1.75				



Max Deflections

Transient Downward	0.074 in	Total Downward	0.101 in
Ratio	1142	Ratio	830
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Email: myengineer@centurytel.net

PROJECT : **4024 85th AVE SE**

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 1008.33\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.69$$

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

$A := t \cdot h$ $A = 30.3\ in^2$

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

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 658.57\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.80$$

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

$A := t \cdot h$ $A = 30.3\ in^2$

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

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

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PROJECT : **4024 85th AVE SE**

Maximum Load For 4-2x6 HF Stud Built up Wood Post

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 756.25\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.64$

$F'_c := C_p \cdot F''_c$ $F'_c = 560\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 18484\ lb$ (Maximum post Capacity)

(4)2x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := (4) \cdot 1.5 \cdot in$

$A := t \cdot h$ $A = 33\ in^2$

$I := \frac{t \cdot h^3}{12}$ $I = 83.2\ in^4$

$S := \frac{I \cdot 2}{h}$ $S = 30.3\ in^3$

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 756.25\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.64$

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

(3)2x6 Wood Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

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PROJECT : **4024 85th AVE SE**

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 756.25\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.64$

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

(2)2x6 Wood Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 306.25\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.32$

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

(3)2x4 Wood Post Properties

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

$h := 3.5 \cdot in$

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

$A := t \cdot h$ $A = 15.8\ in^2$

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

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

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 306.25\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.32$

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

(2)2x4 Wood Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

Maximum Load For 4x4 HF#2 Treated Post

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

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

$E := 1235000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 806.87\ psi$

$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.60$

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

6x6 Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

$A := t \cdot h$ $A = 12.3\ in^2$

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

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

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