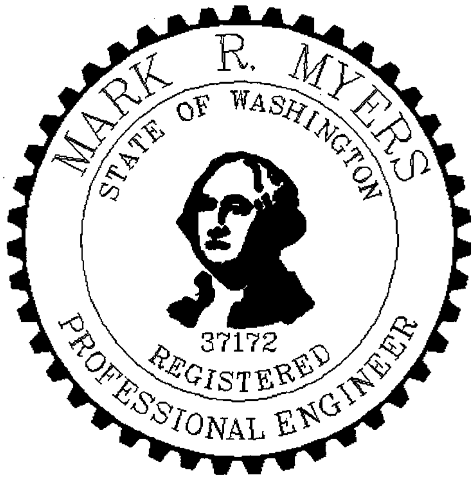
MYERS ENGINEERING

Structural Calculations



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Mark Myers, PE
Date: 2021.10.02
18:20:13 -07'00'

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

Project: Single Family Residence
9026 Southeast 61st Street
Mercer Island, WA

October 2, 2021

2018 INTERNATIONAL BUILDING CODE
100 MPH WIND, EXPOSURE B, K_{zt} = 1.39
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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Gig Harbor, WA 98335
Phone: 253-858-3248
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Myers Engineering, LLC
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Gig Harbor, WA 98335

PROJECT : **9026 SE 61st ST**

Phone: 253-858-3248
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DESIGN LOADS:

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

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

WOODS :

WOOD TYPE:

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

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

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

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

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

TRUSSES:

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

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

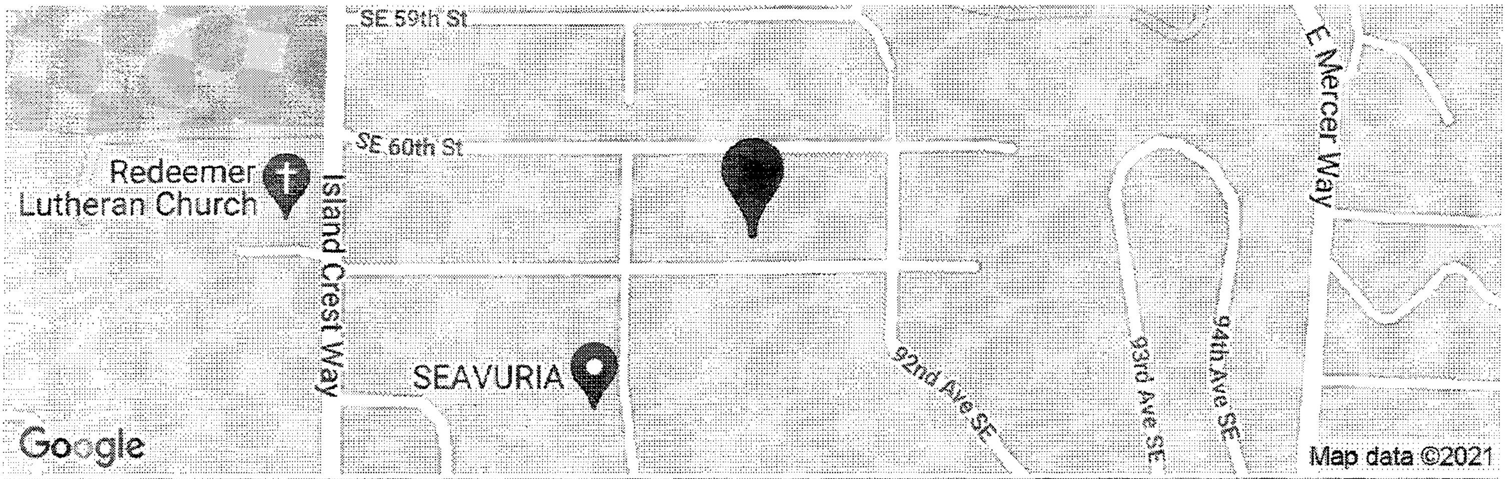
ENGINEERED I-JOISTS

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

OSHPD

9026 SE 61st ST

Latitude, Longitude: 47.5486, -122.2178



Date	9/30/2021, 12:55:42 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.455	MCE_R ground motion. (for 0.2 second period)
S_1	0.504	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.746	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.164	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.623	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.747	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.455	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.613	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	4.269	Factored deterministic acceleration value. (0.2 second)
S1RT	0.504	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.561	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.644	Factored deterministic acceleration value. (1.0 second)
PGAd	1.423	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.899	Mapped value of the risk coefficient at a period of 1 s

LATERAL ANALYSIS :
BASED ON 2018 INTERNATIONAL BUILDING CODE (IBC)

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

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

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

Seismic Design Data:

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

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

$S_s := 1.455$ $\quad S_1 := 0.504$ $\quad S_{ms} := 1.746$ $\quad S_{m1} := 0.907$

Equation 11.4-3 $\quad S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.16$ $\quad$ Equation 11.4-4 $\quad S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.6$

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

Roof Slope Adjustment Factor: $\quad S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{8}{12}\right)\right)} = 1.2$ $\quad S_b := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{4}{12}\right)\right)} = 1.05$

Plan Area for Each Level:

$A_1 := 2240 ft^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1960 ft^2$ (Upper Floor) $\quad A_{2b} := 766 ft^2 \cdot S_b$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(33ft) + 2(61ft)$ (Upper Floor) $\quad P_2 := 2(33ft) + 2(71ft)$ (Main Floor)

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

Weight of Structure at Each Level:

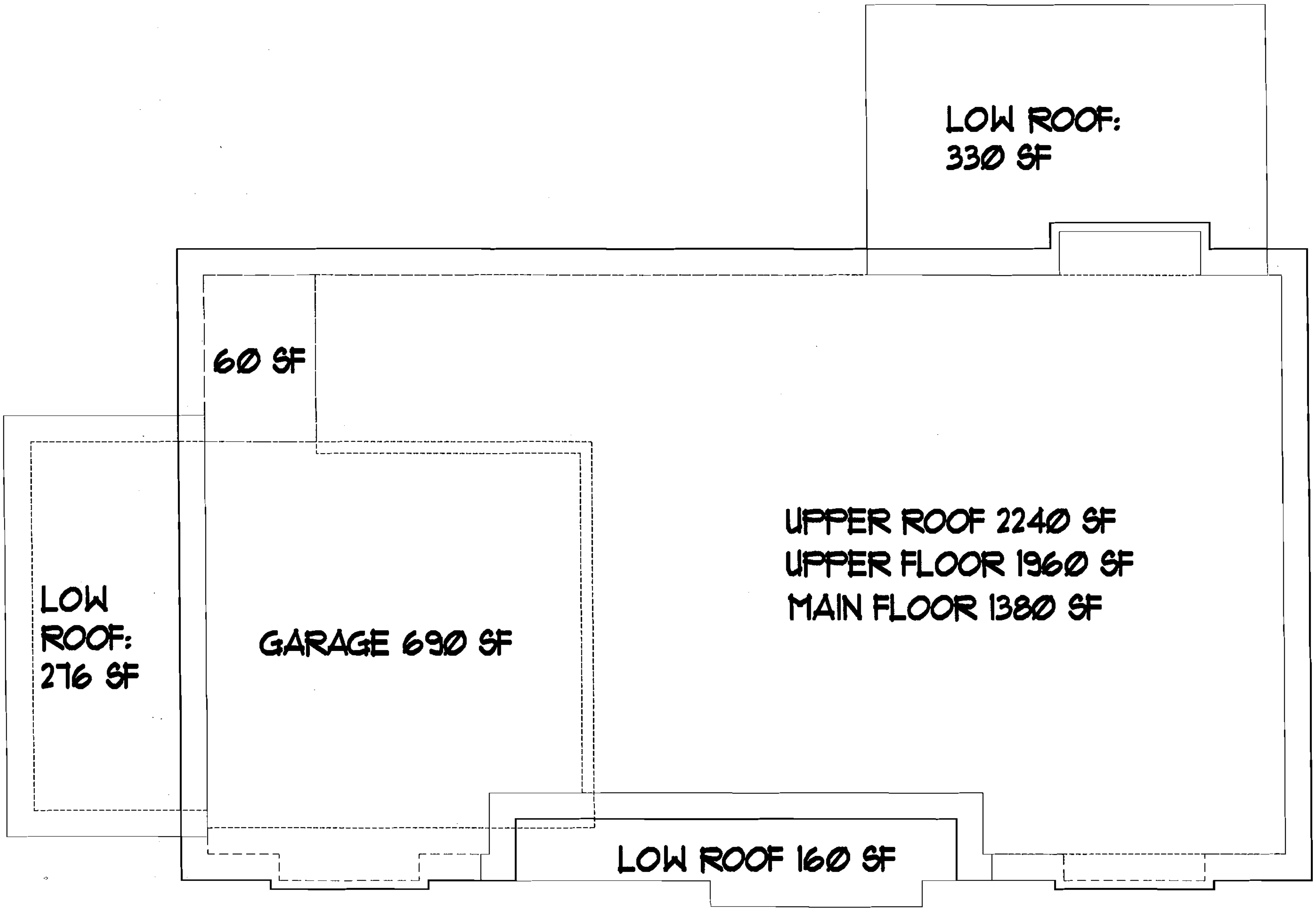
Story Weight at Upper Floor:

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

Story Weight at Main Floor:

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

$$W := w_1 + w_2 = 114677.7\,lb$$


LOW ROOF:
330 SF
60 SF
UPPER ROOF 2240 SF
UPPER FLOOR 1960 SF
MAIN FLOOR 1380 SF
LOW
ROOF:
276 SF
GARAGE 690 SF
LOW ROOF 160 SF

Approximate Fundamental Period, T_a:

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

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

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right) \cdot T_a} = 0.43$ (ASCE 7-16 Eq. 12.8-3)

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

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.18$$ (ASCE 7-16 Eq. 12.8-2)

Total Base Shear: $V_E := C_s \cdot W = 20536.13\,\text{lb}$

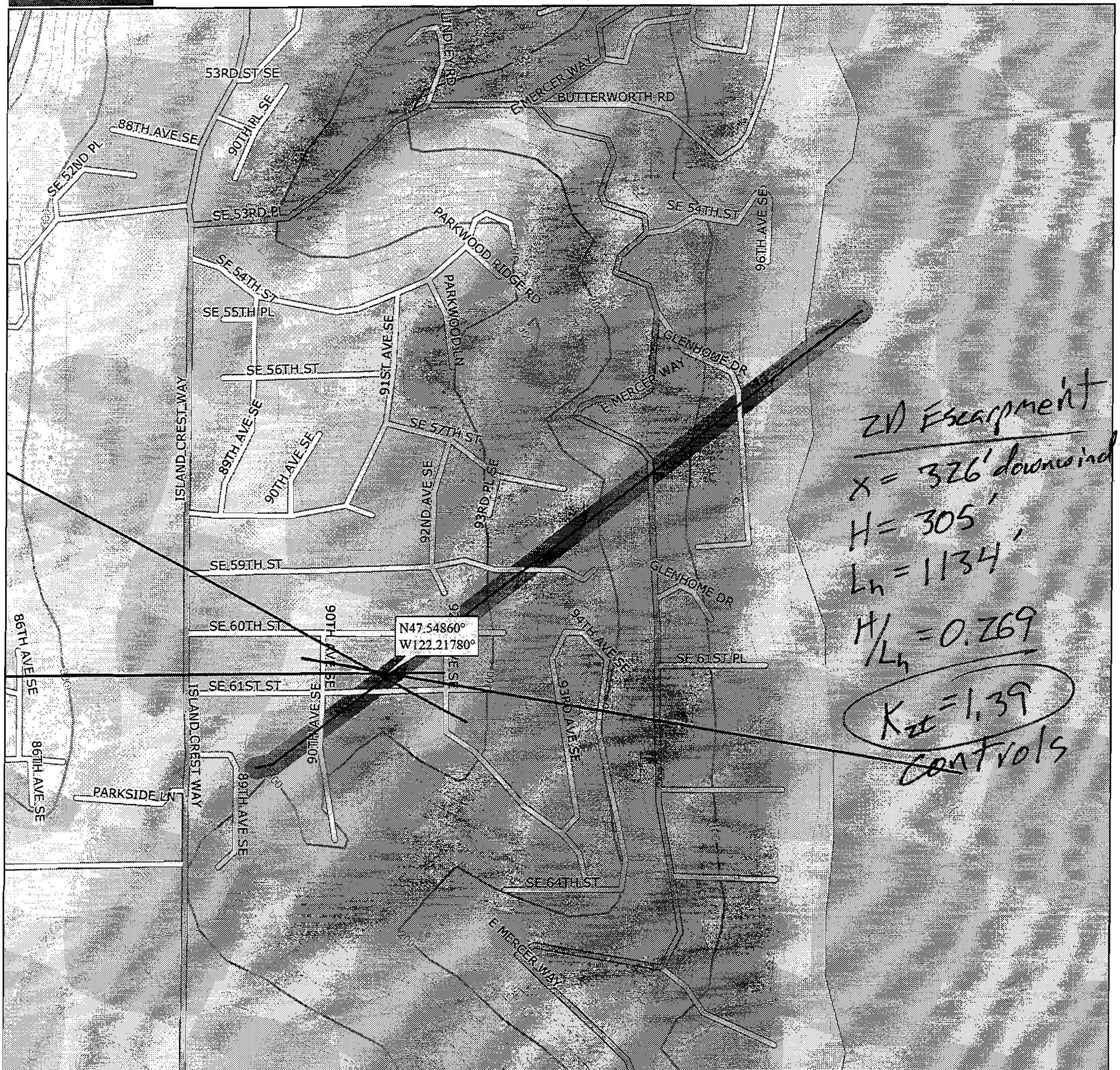
Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

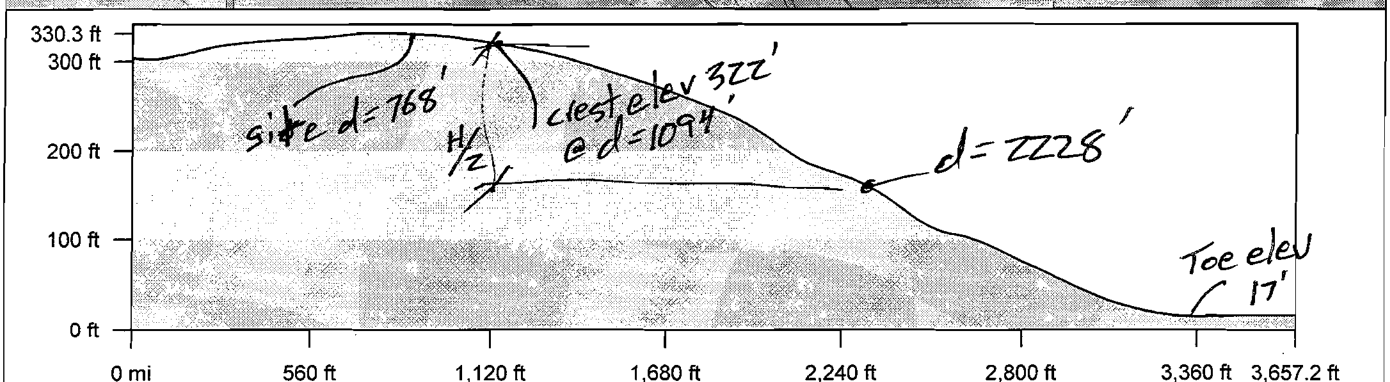
for structures having a period of 0.5 sec or less: $k := 1$

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

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.61$ $F_1 := C_{v1} \cdot V_E = 12562.98\,\text{lb}$ Story Shear at Upper Floor

$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.39$ $F_2 := C_{v2} \cdot V_E = 7973.15\,\text{lb}$ Story Shear at Main Floor

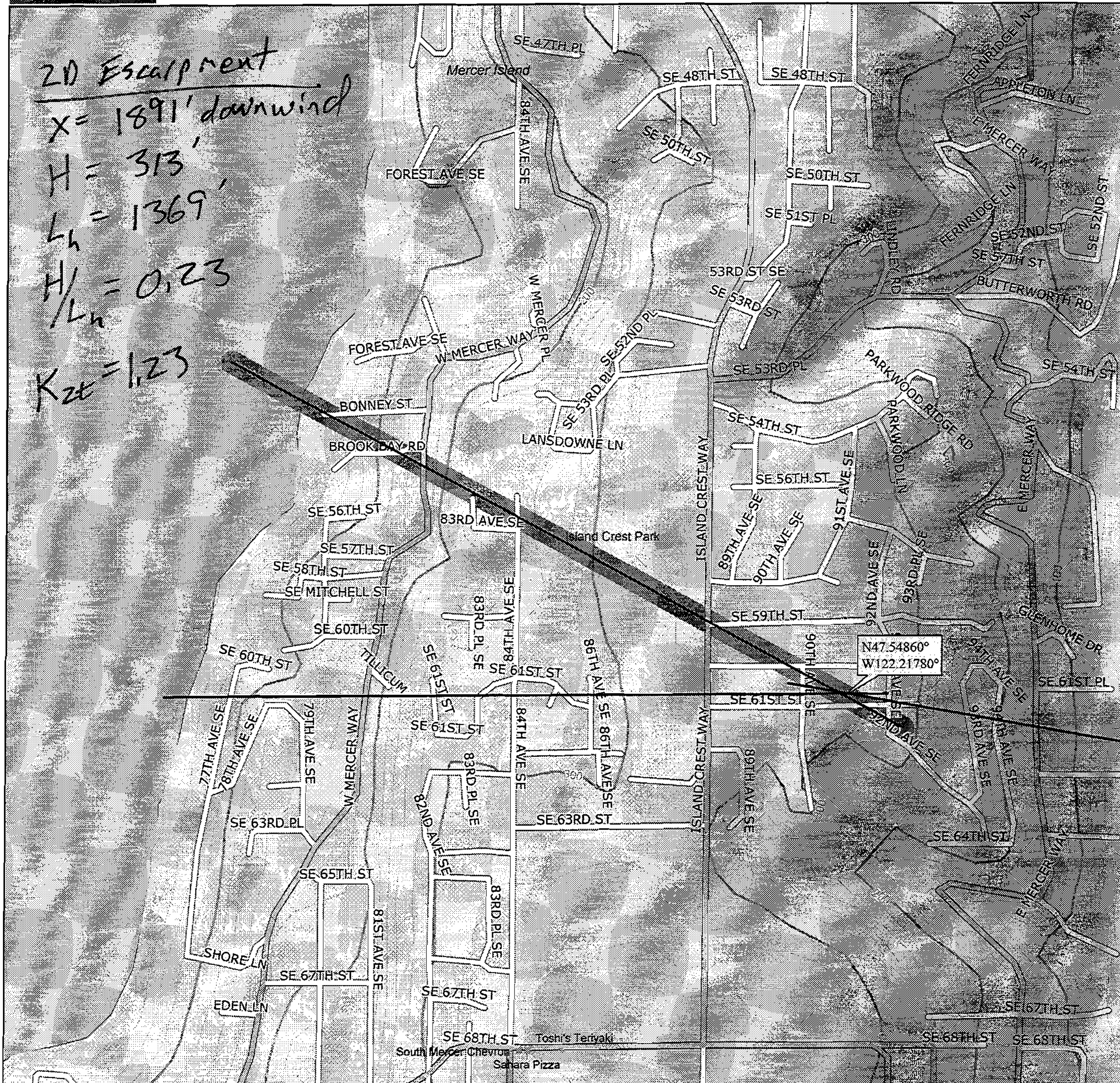


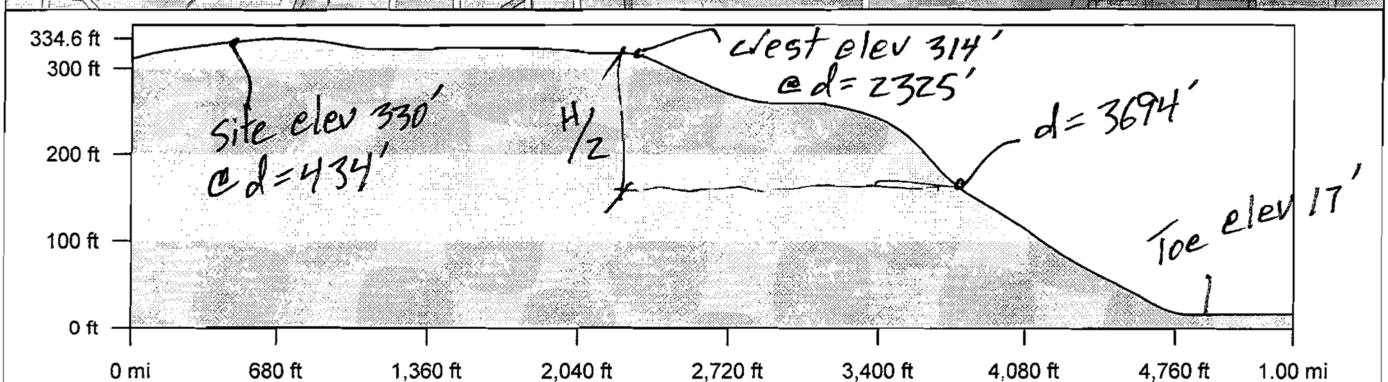


Lin Dist: 3,631.5 ft	Terr Dist: 3,657.2 ft	Elev Gain: -286.9 ft	Avg Grade: 9
Climb Elev: 29.9 ft	Desc Elev: 316.8 ft	Max. Elev: 330.3 ft	Min. Elev: 15.0 ft
Climb Dist: 971.3 ft	Desc Dist: 2,671.8 ft		

TN
MN (14.8°E)

Scale 1 : 8,800
1" = 733.3 ft
Data Zoom 14-5



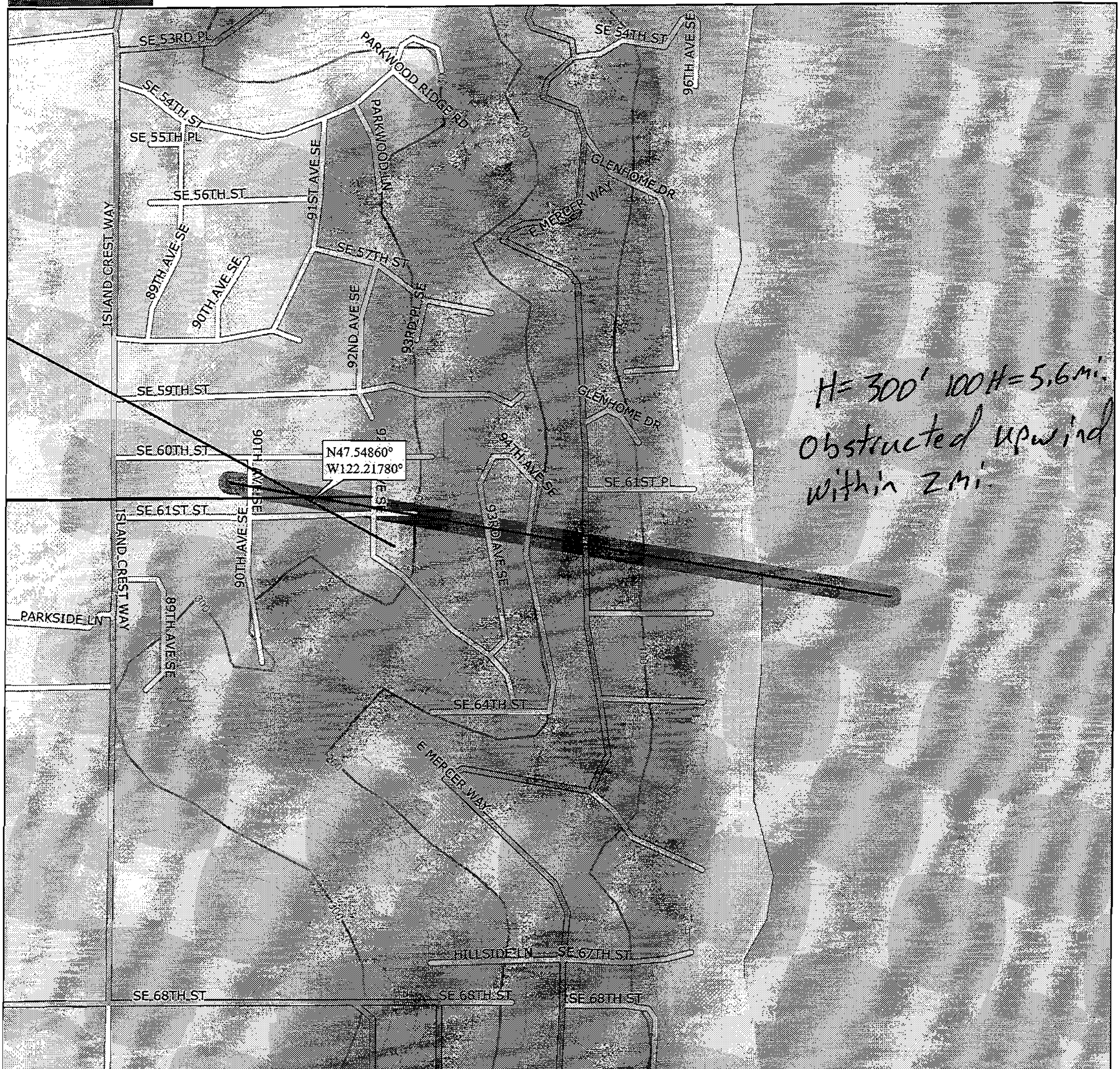


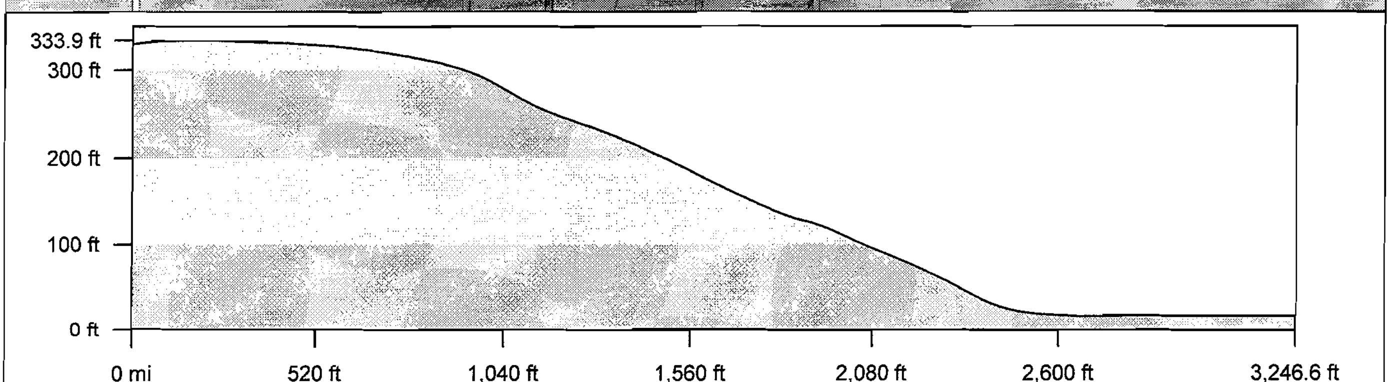
Lin Dist: 5,269.0 ft	Terr Dist: 1.0 mi	Elev Gain: -295.4 ft	Avg Grade: 6
Climb Elev: 27.1 ft	Desc Elev: 322.5 ft	Max. Elev: 334.6 ft	Min. Elev: 15.4 ft
Climb Dist: 1,281.6 ft	Desc Dist: 3,722.3 ft		

TN
MN (14.8°E)

Scale 1 : 12,800
1" = 1,066.7 ft
Data Zoom 14-0

DELORME



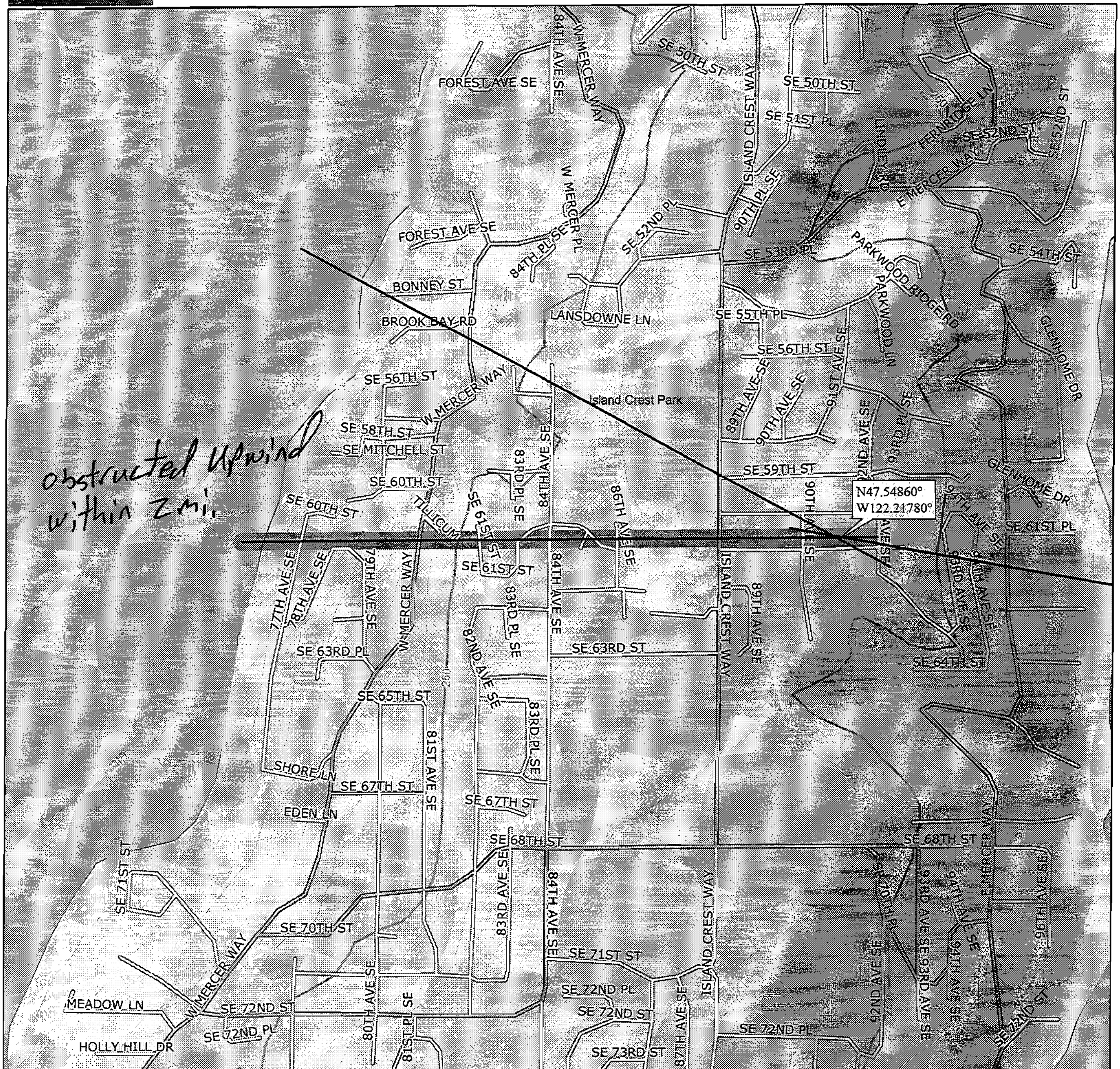


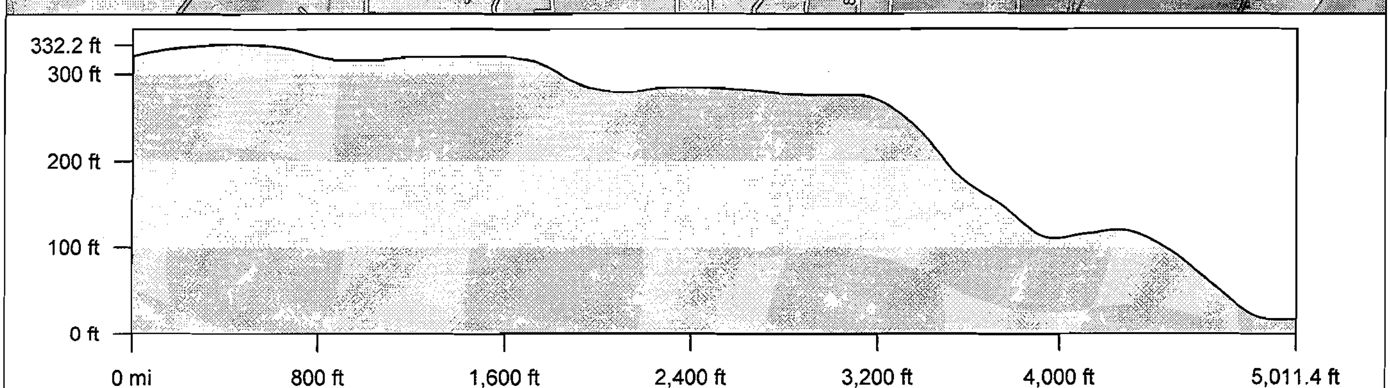
Lin Dist: 3,219.5 ft	Terr Dist: 3,246.6 ft	Elev Gain: -313.3 ft	Avg Grade: 10
Climb Elev: 4.9 ft	Desc Elev: 318.2 ft	Max. Elev: 333.9 ft	Min. Elev: 15.7 ft
Climb Dist: 296.3 ft	Desc Dist: 2,512.5 ft		

TN
MN (14.8°E)

Scale 1 : 8,800

1" = 733.3 ft Data Zoom 14-5





Lin Dist: 4,979.9 ft	Terr Dist: 5,011.4 ft	Elev Gain: -303.6 ft	Avg Grade: 7
Climb Elev: 32.8 ft	Desc Elev: 336.3 ft	Max. Elev: 332.2 ft	Min. Elev: 16.4 ft
Climb Dist: 1,576.3 ft	Desc Dist: 3,435.2 ft		

TN
MN (14.8°E)

Scale 1 : 14,400
1" = 1,200.0 ft   Data Zoom 13-7

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WIND DESIGN

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

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

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

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

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

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

$x := 326ft$ $H := 305ft$ $L_h := 1134ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.75\left(\frac{H}{L_h}\right) = 0.2 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.93 \qquad K_3 := e^{\frac{(-\gamma\cdot z)}{L_h}} = 0.95 \qquad K_{zt} := (1 + K_1\cdot K_2\cdot K_3)^2 = 1.39$$

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

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

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

Velocity Pressure Exposure Coefficient (Table 26.10-1):

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

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

$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.62 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.57 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.66$$

External Pressure Coefficients w/ Roof Pitch = 4.33/12 (20 degrees) Front to Back & 8/12 (34 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 33ft$ $B_{fb} := 61ft$ $\frac{L_{fb}}{B_{fb}} = 0.54$ $\frac{h}{L_{fb}} = 0.73$

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

$C_{pf2} := -0.083$ Windward Roof

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

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

Side to Side:

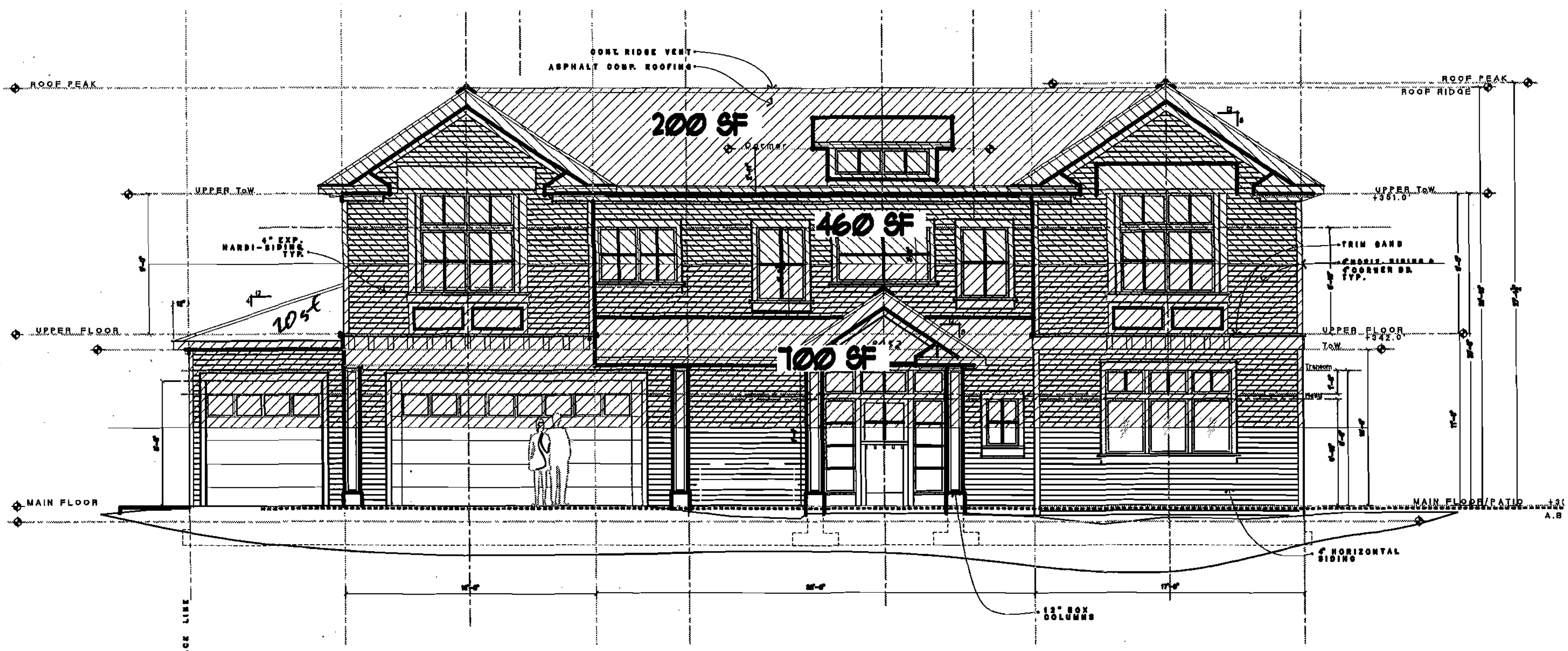
$L_{ss} := 61ft$ $B_{ss} := 33ft$ $\frac{L_{ss}}{B_{ss}} = 1.85$ $\frac{h}{L_{ss}} = 0.39$

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

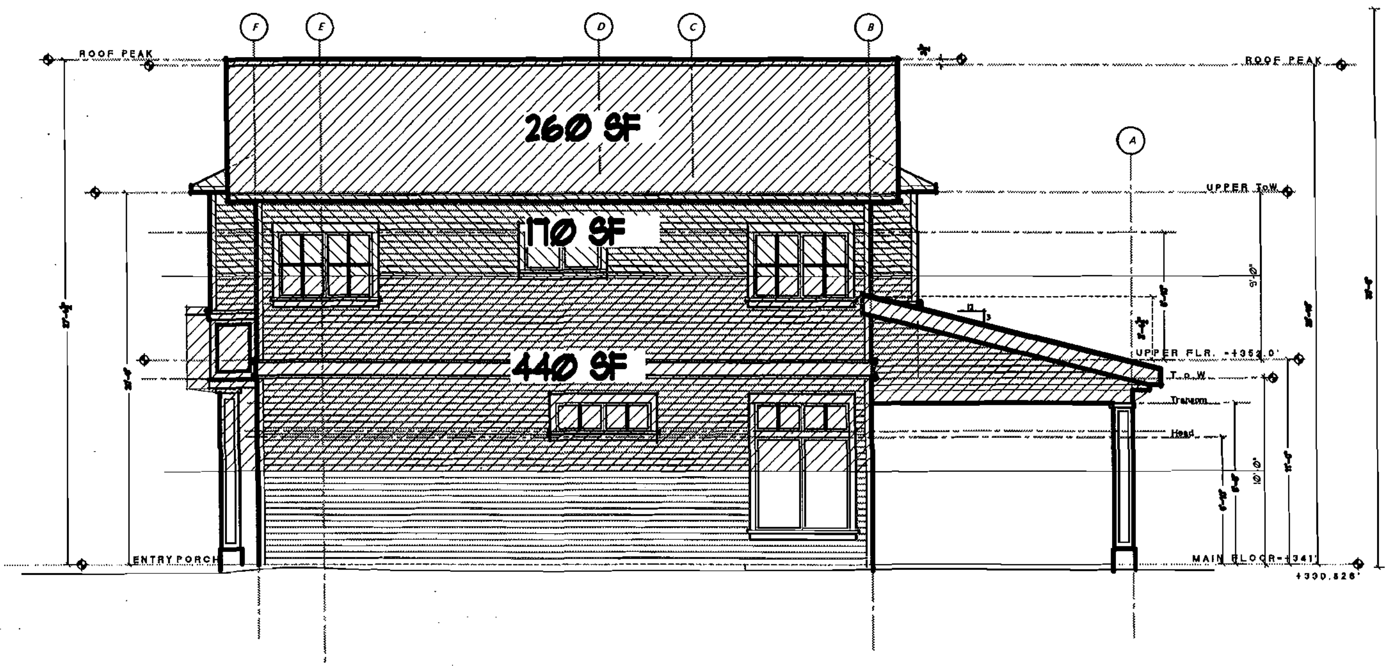
$C_{ps2} := 0.34$ Windward Roof

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

$C_{ps4} := -.33$ Leeward Wall



SOUTH ELEVATION



WEST ELEVATION

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 18.83$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 17.34$ $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 19.83$

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 $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 12.8\,ft^{-2} \cdot lb$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 11.79\,ft^{-2} \cdot lb$

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = -1.4\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -10.12\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -8.43\,ft^{-2} \cdot lb$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 5.73\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -10.12\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -5.56\,ft^{-2} \cdot lb$

The Internal Pressures on Windward and Leeward Walls & Roofs will offset each other for the lateral design of the overall building and will therefore be ignored for this application.

Check net pressure not less than 16psf at walls & 8psf at roof over projected vertical plane per ASCE 7-16 Sec. 27.1-5:

$p_{wr1} - p_{lr1} = 8.72\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw1} = 21.23\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw1} = 20.22\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 15.85\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw2} = 18.37\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw2} = 17.36\,ft^{-2} \cdot lb$

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

$$V_{1W} := (p_{wr1} - p_{lr1})200ft^2 + (p_{ww1} - p_{lw1}) \cdot 460 \cdot ft^2 = 11509.95\,lb$$

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

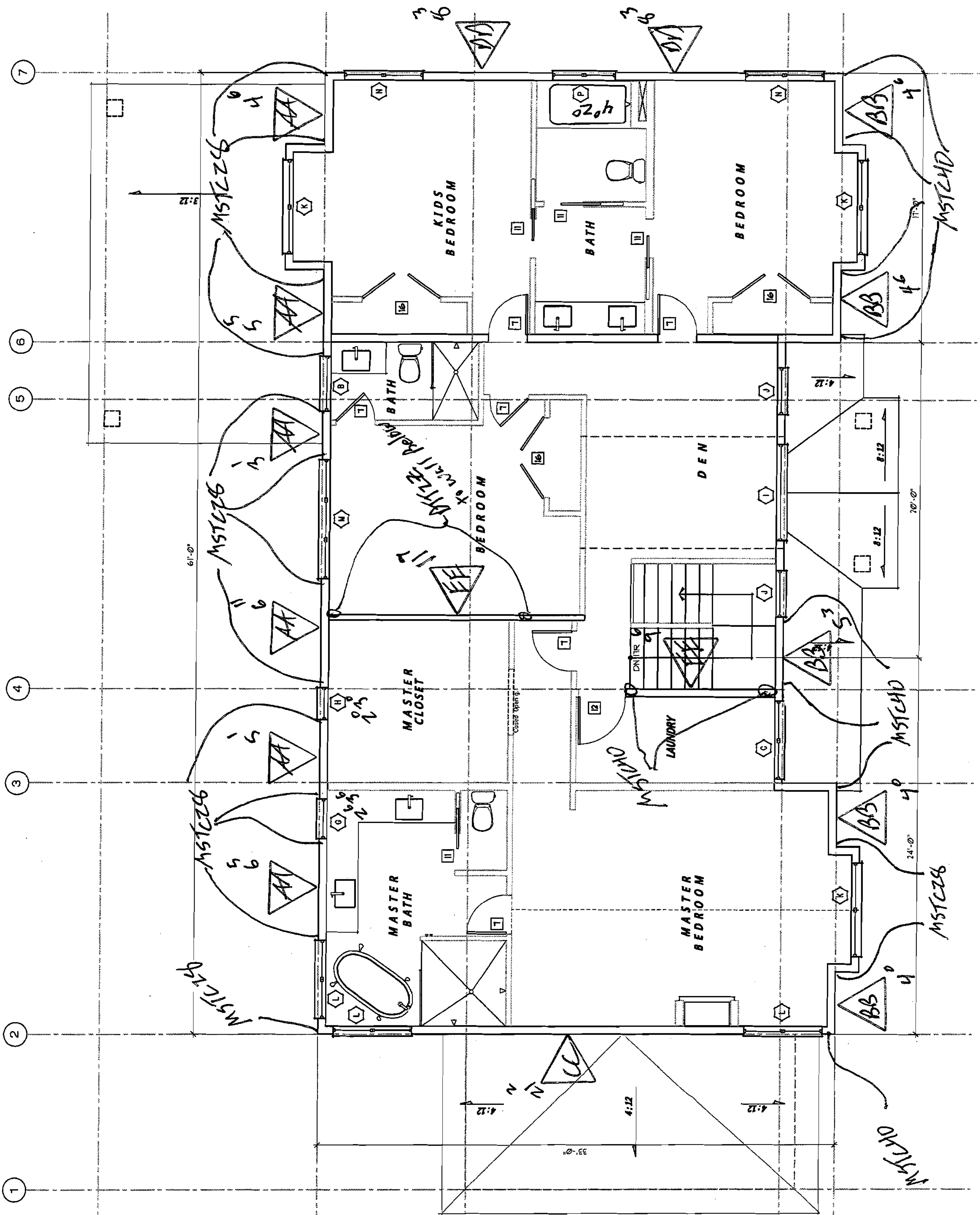
$$V_{2W} := (p_{wr1} - p_{lr1})20ft^2 + (p_{ww2} - p_{lw1}) \cdot 700 \cdot ft^2 = 14329.59\,lb$$

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

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 260ft^2 + (p_{ww1} - p_{lw2}) \cdot 170ft^2 = 7242.54\,lb$$

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

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 440ft^2 = 7636.55\,lb$$


KIDS BEDROOM
BATH
BEDROOM
BATH
BEDROOM
DEN
MASTER CLOSET
LAUNDRY
MASTER BATH
MASTER BEDROOM
Cased Opening
DN
UP
61'-0"
20'-0"
24'-0"
17'-0"
33'-0"
3:12
4:12
8:12
MSTCZ26
MSTC40
AA
BB
CC
DD
EE
FF

13

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PROJECT : **9026 SE 61st ST**

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WALL AA:

Story Shear due to Wind: $V_{3W} = 7242.54\,lb$ Story Shear due to Seismic: $F_1 = 12562.98\,lb$

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

Shear Wall Length: $Laa := \left[6.42 + 5.08 + 6.92 + 3.08\left(\frac{6.17}{9}\right) + 5.42 + 4.5\right]ft = 30.45\,ft$

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

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

$vaa = 71.35\,ft^{-1} \cdot lb$ $\frac{vaa}{C_o} = 71.35\,ft^{-1} \cdot lb$ $E_{aa} = 144.39\,ft^{-1} \cdot lb$ $\frac{E_{aa}}{C_o} = 144.39\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{aa} := 3.08 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{aa} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRaa := \frac{W_{aa} \cdot L_{aa}}{2}$ $DLRaa = 184.8\,lb$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_w = 642.16\,lb$ $CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}}$ $CFaa_s = 1299.55\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 531.28\,lb$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 1218.79\,lb$

Simpson MSTC28

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

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

$B_p := \frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 2.29\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 1.13\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vaa} = 19.28\,ft$ $\frac{(Z_B \cdot C_o)}{E_{aa}} = 9.53\,ft$

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

WALL BB:

Story Shear due to Wind: $V_{3W} = 7242.54\,lb$

Story Shear due to Seismic: $F_1 = 12562.98\,lb$

Bldg Width in direction of Load: $L_t := 33 \cdot ft$

Distance between shear walls: $L_1 := 33 \cdot ft$

Shear Wall Length: $Lbb := \left[2 \cdot 4\left(\frac{8}{9}\right) + 5.25 + 2 \cdot 4.5\right] ft = 21.36\,ft$

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

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

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

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

$vbb = 101.72\,ft^{-1} \cdot lb$ $\quad \frac{vbb}{C_o} = 101.72\,ft^{-1} \cdot lb$

$E_{bb} = 205.84\,ft^{-1} \cdot lb$ $\quad \frac{E_{bb}}{C_o} = 205.84\,ft^{-1} \cdot lb$

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

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

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

$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $\quad DLRbb = 240\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\quad CFbb_w = 915.44\,lb$

$CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}}$ $\quad CFbb_s = 1852.59\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 771.44\,lb$

$HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS}) \cdot DLRbb = 1747.7\,lb$

Simpson MSTC40 at wall or MSTC28 at flush beam (LSTHD8 at foundation)

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

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

$B_P := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 1.6\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.79\,ft$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vbb} = 13.53\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_{bb}} = 6.68\,ft$

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 11509.95\,lb$

Story Shear due to Seismic: $F_1 = 12562.98\,lb$

Bldg Width in direction of Load: $L_t := 61\cdot ft$

Distance between shear walls: $L_1 := 24\cdot ft$

Shear Wall Length: $Lcc := (21.17)ft = 21.17\,ft$

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

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

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

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

$vcc = 64.17\,ft^{-1}\cdot lb$ $\quad \frac{vcc}{C_o} = 64.17\,ft^{-1}\cdot lb$

$E_{cc} = 81.72\,ft^{-1}\cdot lb$ $\quad \frac{E_{cc}}{C_o} = 81.72\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{cc} := 21.17\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{cc} := (15\cdot psf)\cdot 9\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRcc := \frac{W_{cc}\cdot L_{cc}}{2}$ $\quad DLRcc = 2381.63\,lb$

Chord Force:

$CFcc_w := \frac{vcc\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $\quad CFcc_w = 577.56\,lb$

$CFcc_s := \frac{E_{cc}\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $\quad CFcc_s = 735.47\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = -851.41\,lb$

$HDFcc_s := CFcc_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRcc = -305.4\,lb$

No Holdown Required

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

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

$B_R := \frac{\left(C_D\cdot Z_N\cdot C_o\right)}{vcc} = 2.54\,ft$ $\quad \frac{\left(C_D\cdot Z_N\cdot C_o\right)}{E_{cc}} = 2\,ft$

16d @ 8" o.c.

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

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

$As := \frac{\left(Z_B\cdot C_o\right)}{vcc} = 21.44\,ft$ $\quad \frac{\left(Z_B\cdot C_o\right)}{E_{cc}} = 16.84\,ft$

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

WALL DD:

Story Shear due to Wind: $V_{1W} = 11509.95\,lb$

Story Shear due to Seismic: $F_1 = 12562.98\,lb$

Bldg Width in direction of Load: $L_t := 61 \cdot ft$

Distance between shear walls: $L_1 := 37 \cdot ft$

Shear Wall Length: $Ldd := (2 \cdot 8.25)ft = 16.5\,ft$

Percent full height sheathing: $\% := \left(\frac{16.5 \cdot ft}{20.5 \cdot ft}\right) \cdot 100$ $\quad \% = 80.49$

Max Opening Height = 2ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

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

Seismic Force: $\rho := 1.0$ $\quad E_{dd} := \frac{\rho \cdot \frac{0.7F_1}{L_t} \cdot \frac{L_1}{2}}{Ldd}$

$vdd = 126.94\,ft^{-1} \cdot lb$ $\quad \frac{vdd}{C_o} = 126.94\,ft^{-1} \cdot lb$

$E_{dd} = 161.64\,ft^{-1} \cdot lb$ $\quad \frac{E_{dd}}{C_o} = 161.64\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_{dd} := 16.5 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

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

$DLRdd := \frac{W_{dd} \cdot L_{dd}}{2}$ $\quad DLRdd = 1980\,lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $\quad CFdd_w = 1142.42\,lb$

$CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}}$ $\quad CFdd_s = 1454.76\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = -45.58\,lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 589.42\,lb$

No Holdown Required

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

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

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vdd} = 1.29\,ft$ $\quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 1.01\,ft$

16d @ 12" o.c.

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

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

$A_s := \frac{(Z_B \cdot C_o)}{vdd} = 10.84\,ft$ $\quad \frac{(Z_B \cdot C_o)}{E_{dd}} = 8.51\,ft$

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

WALL EE:

Story Shear due to Wind: $V_{3W} = 7242.54\,\text{lb}$ Story Shear due to Seismic: $F_1 = 12562.98\,\text{lb}$

Bldg Width in direction of Load: $L_t := 61\cdot\text{ft}$ Distance between shear walls: $L_1 := 24\cdot\text{ft}$ $L_2 := 37\text{ft}$

Shear Wall Length:

$$Lee := (9.5 + 11.58)\text{ft} = 21.08\,\text{ft}$$

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

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

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

$vee = 103.07\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{vee}{C_o} = 103.07\,\text{ft}^{-1}\cdot\text{lb}$ $E_{ee} = 208.59\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{E_{ee}}{C_o} = 208.59\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_{ee} := 11.58\cdot\text{ft}$ Plate Height: $Pt := 9\cdot\text{ft}$

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

$DLRee := \dfrac{W_{ee}\cdot L_{ee}}{2}$ $DLRee = 694.8\,\text{lb}$

Chord Force:

$CFee_w := \dfrac{vee\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_w = 927.65\,\text{lb}$ $CFee_s := \dfrac{E_{ee}\cdot L_{ee}\cdot Pt}{C_o\cdot L_{ee}}$ $CFee_s = 1877.29\,\text{lb}$

Holdown Force:

$HDFee_w := CFee_w - 0.6\cdot DLRee = 510.77\,\text{lb}$ $HDFee_s := CFee_s - (0.6 - 0.14S_{DS})DLRee = 1573.64\,\text{lb}$

Simpson MSTC40 or DTT2Z to inverted DTT2Z

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

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

$B_N := \dfrac{(Z_N\cdot C_D\cdot C_o)}{vee} = 1.58\,\text{ft}$ $\dfrac{(C_D\cdot Z_N\cdot C_o)}{E_{ee}} = 0.78\,\text{ft}$

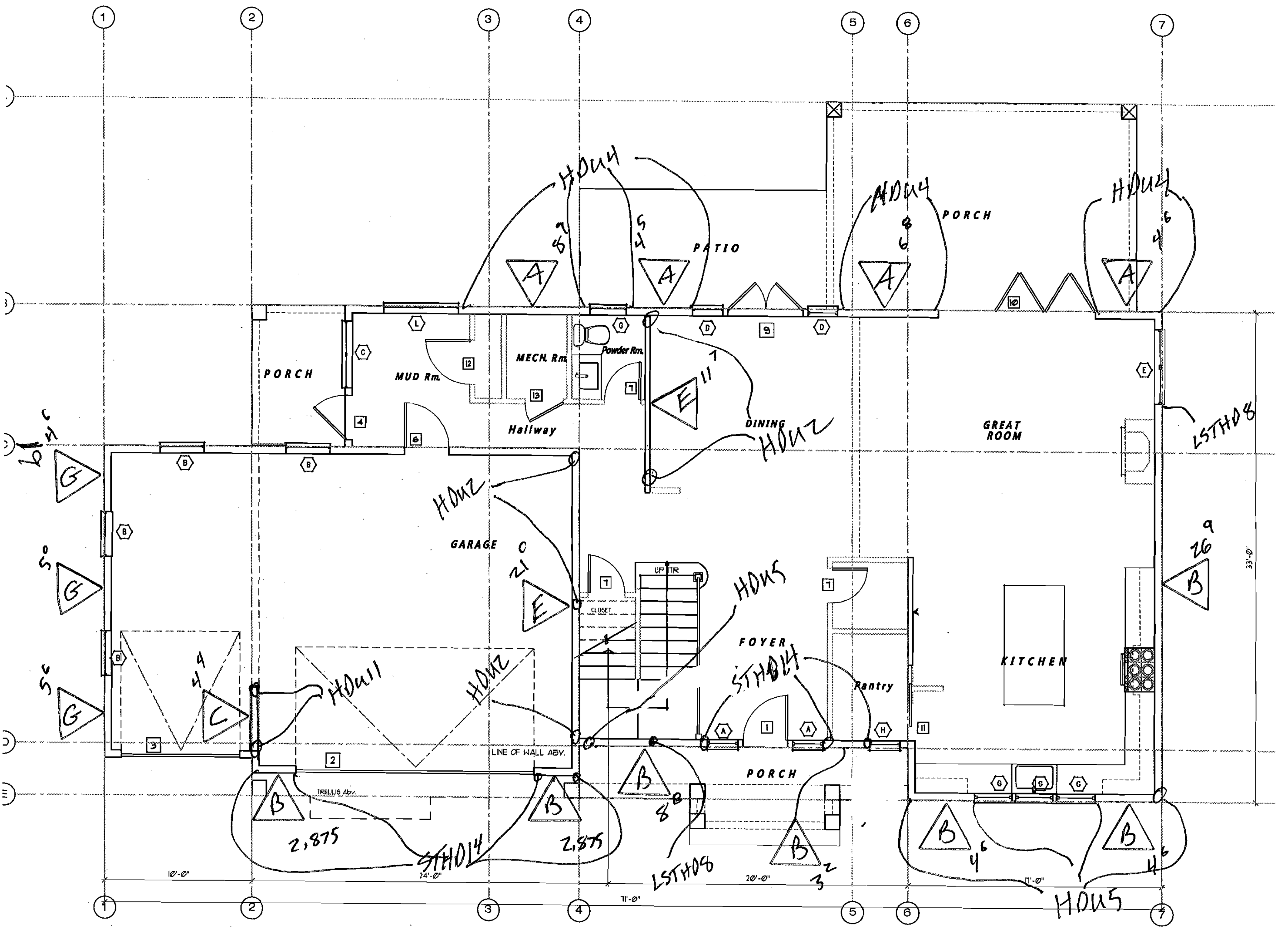
16d @ 8" o.c.

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

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

$As := \dfrac{(Z_B\cdot C_o)}{vee} = 13.35\,\text{ft}$ $\dfrac{(Z_B\cdot C_o)}{E_{ee}} = 6.6\,\text{ft}$

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



PORCH
MUD Rm.
MECH. Rm.
Powder Rm.
Hallway
PATIO
PORCH
DINING
GREAT ROOM
GARAGE
CLOSET
UP 11R
FOYER
Pantry
KITCHEN
PORCH
LINE OF WALL ABV.
TRELLIS Abv.
10'-0"
24'-0"
11'-0"
20'-0"
17'-0"
33'-0"
HDU4
HDU2
HDU5
HDU11
STHD14
LSTHD8
2,875

WALL A:

Story Shear due to Wind: $V_{4W} = 7636.55\,lb$ Story Shear due to Seismic: $F_2 = 7973.15\,lb$

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

Shear Wall Length: $La := \left[8.75 + 4.42\left(\frac{8.83}{10}\right) + 6.67 + 4.5\left(\frac{9}{10}\right)\right]ft = 23.37\,ft$

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

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

$va = 190.98\,ft^{-1}\cdot lb$ $\frac{va}{C_o} = 190.98\,ft^{-1}\cdot lb$ $E_a = 307.52\,ft^{-1}\cdot lb$ $\frac{E_a}{C_o} = 307.52\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_a := 4.42\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_a := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 1\,ft$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 243.1\,lb$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 1909.79\,lb$ $CFa_w + CFaa_w = 2551.95\,lb$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 3075.21\,lb$ $CFa_s + CFaa_s = 4374.76\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 1763.93\,lb$ $HDFa_w + HDFaa_w = 2295.21\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 2968.97\,lb$ $HDFa_s + HDFaa_s = 4187.75\,lb$

Simpson HDU4 w/ SB5/8x24 anchor

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.85\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 0.53\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{va} = 7.2\,ft$ $\frac{(Z_B\cdot C_o)}{E_a} = 4.47\,ft$

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

WALL B:

Story Shear due to Wind: $V_{4W} = 7636.55\,\text{lb}$ Story Shear due to Seismic: $F_2 = 7973.15\,\text{lb}$

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

Shear Wall Length: $Lb := \left[2\cdot 2.875\left(\frac{5.75}{10}\right) + 8.67 + 3.17\left(\frac{6.33}{10}\right) + 2\cdot 4.5\left(\frac{9}{10}\right)\right]\text{ft} = 22.08\,\text{ft}$

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

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

$vb = 202.14\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{vb}{C_o} = 202.14\,\text{ft}^{-1}\cdot\text{lb}$ $E_b = 325.49\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_b}{C_o} = 325.49\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_b := 3.17\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_b := (15\cdot\text{psf})\cdot 2\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 0\text{ft}$ $DLRb := \frac{W_b\cdot L_b}{2}$ $DLRb = 206.05\,\text{lb}$

Chord Force:

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_w = 2021.35\,\text{lb}$ $CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_s = 3254.85\,\text{lb}$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 1897.72\,\text{lb}$ $HDFb_s := CFb_s - (0.6 - 0.14S_{DS})\cdot DLRb = 3164.8\,\text{lb}$

Simpson STHD14/RJ

$HDFb_w + HDFbb_w = 2669.17\,\text{lb}$ $HDFb_s + HDFbb_s = 4912.5\,\text{lb}$

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 0.81\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.5\,\text{ft}$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vb} = 6.81\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_b} = 4.23\,\text{ft}$

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

Myers Engineering, LLC
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PROJECT : **9026 SE 61st ST**

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

WALL C:

Story Shear due to Wind: $V_{2W} = 14329.59\,lb$ Story Shear due to Seismic: $F_2 = 7973.15\,lb$

Bldg Width in direction of Load: $L_t := 71\cdot ft$ Distance between shear walls: $L_1 := 24ft$ $L_2 := 10ft$

Shear Wall Length: $Lc := \left[4.33\left(\frac{8.67}{10}\right)\right]ft = 3.75\,ft$

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

Wind Force: $vc := \frac{vcc\cdot Lcc + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lc}$ Seismic Force: $\rho := 1.3$ $E_c := \frac{E_{cc}\cdot Lcc + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Lc}$

$vc = 910.25\,ft^{-1}\cdot lb$ $\frac{vc}{C_o} = 910.25\,ft^{-1}\cdot lb$ $E_c = 923.58\,ft^{-1}\cdot lb$ $\frac{E_c}{C_o} = 923.58\,ft^{-1}\cdot lb$

P2-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
BOTH SIDES
Wind Capacity = 1372 plf
Seismic Capacity = 980 plf

Dead Load Resisting Overturning: $L_c := 4.33\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_c := (15\cdot psf)\cdot 1\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 11ft$ $DLRc := \frac{W_c\cdot L_c}{2}$ $DLRc = 487.12\,lb$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_w = 9102.47\,lb$ $CFc_w + CFcc_w = 9680.03\,lb$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c}$ $CFc_s = 9235.83\,lb$ $CFc_s + CFcc_s = 9971.3\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 8810.2\,lb$ $HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 9022.94\,lb$

Simpson HDU11 at 3.5"x5.5" DF post minimum w/ PAB8 anchor embedded 8" into 24" wide footing

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

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

$B_N := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 0.18\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 0.18\,ft$

Not Applicable

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

$A_s := 1070\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1712\,lb$

$A_s := \frac{(Z_B\cdot C_o)}{vc} = 1.88\,ft$ $\frac{(Z_B\cdot C_o)}{E_c} = 1.85\,ft$

5/8" A.B. @ 18" o.c.

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PROJECT : **9026 SE 61st ST**

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WALL D:

Story Shear due to Wind: $V_{2W} = 14329.59\,lb$

Story Shear due to Seismic: $F_2 = 7973.15\,lb$

Bldg Width in direction of Load: $L_t := 71\cdot ft$

Distance between shear walls: $L_1 := 37\cdot ft$

Shear Wall Length: $Ld := (26.75)ft = 26.75\,ft$

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

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

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

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

$vd = 162.04\,ft^{-1}\cdot lb$ $\quad \frac{vd}{C_o} = 162.04\,ft^{-1}\cdot lb$

$E_d = 154.07\,ft^{-1}\cdot lb$ $\quad \frac{E_d}{C_o} = 154.07\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_d := 26.75\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_d := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 8.5ft$

$DLRd := \frac{W_d\cdot L_d}{2}$ $\quad DLRd = 2474.37\,lb$

Chord Force:

$CFd_w := \frac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $\quad CFd_w = 1620.45\,lb$

$CFd_w + CFdd_w = 2762.86\,lb$

$CFd_s := \frac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $\quad CFd_s = 1540.68\,lb$

$CFd_s + CFdd_s = 2995.44\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 135.82\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 459.28\,lb$

$HDFd_w + HDFdd_w = 90.24\,lb$

$HDFd_s + HDFdd_s = 1048.7\,lb$

Simpson LSTHD8

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

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

$B_P := \frac{(C_D\cdot Z_N\cdot C_o)}{vd} = 1.01\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 1.06\,ft$

16d @ 12" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vd} = 8.49\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_d} = 8.93\,ft$

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

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WALL E:

Story Shear due to Wind: $V_{2W} = 14329.59\,\text{lb}$ Story Shear due to Seismic: $F_2 = 7973.15\,\text{lb}$

Bldg Width in direction of Load: $L_t := 71\cdot\text{ft}$ Distance between shear walls: $L_1 := 24\cdot\text{ft}$ $L_2 := 37\text{ft}$

Shear Wall Length: $Le := (21 + 11.58)\text{ft} = 32.58\,\text{ft}$

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

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

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

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

$ve = 180.05\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{ve}{C_o} = 180.05\,\text{ft}^{-1}\cdot\text{lb}$

$E_e = 208.55\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{E_e}{C_o} = 208.55\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_e := 11.58\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

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

$DLRe := \dfrac{W_e\cdot L_e}{2}$ $DLRe = 1563.3\,\text{lb}$

Chord Force:

$CFe_w := \dfrac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 1800.54\,\text{lb}$

$CFe_s := \dfrac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 2085.51\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 862.56\,\text{lb}$

$HDFe_w + HDFee_w = 1373.33\,\text{lb}$

$HDFe_s := CFe_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRe = 1402.29\,\text{lb}$

$HDFe_s + HDFee_s = 2975.93\,\text{lb}$

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

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

$B_R := \dfrac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.91\,\text{ft}$ $\dfrac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.78\,\text{ft}$

16d @ 8" o.c.

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

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

$As := \dfrac{(Z_B\cdot C_o)}{ve} = 7.64\,\text{ft}$ $\dfrac{(Z_B\cdot C_o)}{E_e} = 6.6\,\text{ft}$

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

WALL G:

Story Shear due to Wind: $V_{2W} = 14329.59\,lb$ Story Shear due to Seismic: $F_2 = 7973.15\,lb$

Bldg Width in direction of Load: $L_t := 71\cdot ft$ Distance between shear walls: $L_1 := 10\cdot ft$

Shear Wall Length: $Lg := \left[5.5 + 5 + 4.5\left(\frac{9}{10}\right)\right]ft = 14.55\,ft$

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

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

Wind Force: $vg := \dfrac{\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}}{Lg}$ Seismic Force: $\rho := 1.0$ $E_g := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}}{Lg}$

$vg = 41.61\,ft^{-1}\cdot lb$ $\frac{vg}{C_o} = 41.61\,ft^{-1}\cdot lb$ $E_g = 27.01\,ft^{-1}\cdot lb$ $\frac{E_g}{C_o} = 27.01\,ft^{-1}\cdot lb$

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

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

$W_g := (15\cdot psf)\cdot 4\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRg := \frac{W_g\cdot L_g}{2}$ $DLRg = 360\,lb$

Chord Force:

$CFg_w := \frac{vg\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_w = 416.13\,lb$ $CFg_s := \frac{E_g\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_s = 270.13\,lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6\cdot DLRg = 200.13\,lb$ $HDFg_s := CFg_s - (0.6 - 0.14S_{DS})\cdot DLRg = 112.8\,lb$

No Holdown Required

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vg} = 3.92\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_g} = 6.04\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vg} = 33.07\,ft$ $\frac{(Z_B\cdot C_o)}{E_g} = 50.94\,ft$

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

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Diapragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa\cdot\frac{Laa}{61ft} = 35.62\,ft^{-1}\cdot lb \qquad E_{aa}\cdot\frac{Laa}{61ft} = 72.08\,ft^{-1}\cdot lb$$

Wall Lines DD:

$$vdd\cdot\frac{Ldd}{33ft} = 63.47\,ft^{-1}\cdot lb \qquad E_{dd}\cdot\frac{Ldd}{33ft} = 80.82\,ft^{-1}\cdot lb$$

Wall Lines BB:

$$vbb\cdot\frac{Lbb}{61ft} = 35.62\,ft^{-1}\cdot lb \qquad E_{bb}\cdot\frac{Lbb}{61ft} = 72.08\,ft^{-1}\cdot lb$$

Wall Lines EE:

$$vee\cdot\frac{Lee}{29ft} = 74.92\,ft^{-1}\cdot lb \qquad E_{ee}\cdot\frac{Lee}{29ft} = 151.62\,ft^{-1}\cdot lb$$

Wall Lines CC:

$$vcc\cdot\frac{Lcc}{33ft} = 41.17\,ft^{-1}\cdot lb \qquad E_{cc}\cdot\frac{Lcc}{33ft} = 52.42\,ft^{-1}\cdot lb$$

Wall Lines A:

$$\frac{va\cdot La - vaa\cdot Laa}{61ft} = 37.56\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La - E_{aa}\cdot Laa}{61ft} = 45.75\,ft^{-1}\cdot lb \qquad \frac{va\cdot La}{61ft} = 73.18\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La}{61ft} = 117.83\,ft^{-1}\cdot lb$$

Wall Lines B:

$$\frac{vb\cdot Lb - vbb\cdot Lbb}{53ft} = 43.23\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb - E_{bb}\cdot Lbb}{53ft} = 52.65\,ft^{-1}\cdot lb \qquad \frac{vb\cdot Lb}{53ft} = 84.22\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb}{53ft} = 135.62\,ft^{-1}\cdot lb$$

Wall Lines C:

$$\frac{vc\cdot Lc - vcc\cdot Lcc}{33ft} = 62.38\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc - E_{cc}\cdot Lcc}{33ft} = 52.64\,ft^{-1}\cdot lb \qquad \frac{vc\cdot Lc}{33ft} = 103.55\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc}{33ft} = 105.07\,ft^{-1}\cdot lb$$

Wall Lines D:

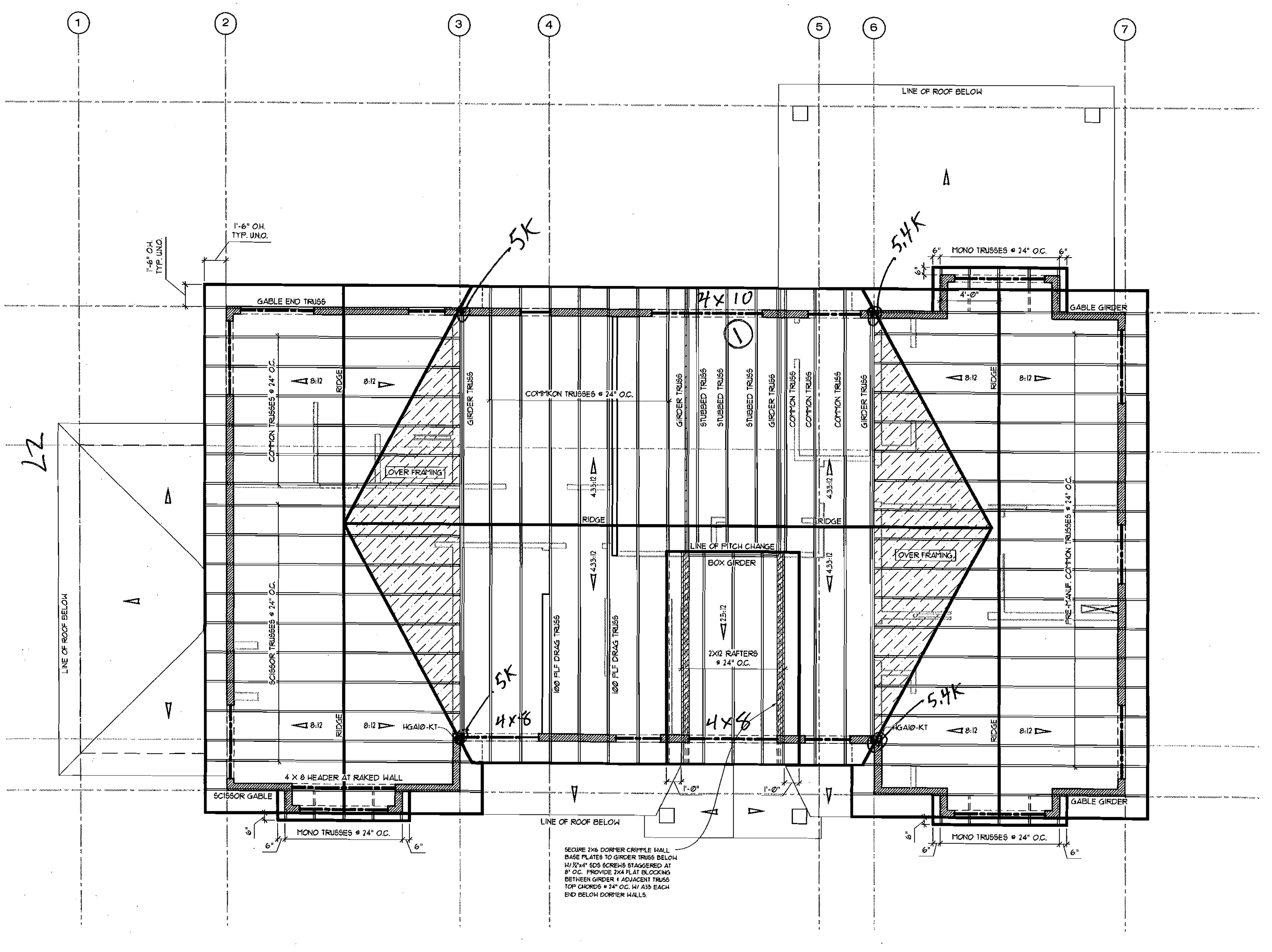
$$\frac{vd\cdot Ld - vdd\cdot Ldd}{33ft} = 67.89\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld - E_{dd}\cdot Ldd}{33ft} = 44.07\,ft^{-1}\cdot lb \qquad \frac{vd\cdot Ld}{33ft} = 131.35\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld}{33ft} = 124.89\,ft^{-1}\cdot lb$$

Wall Line E:

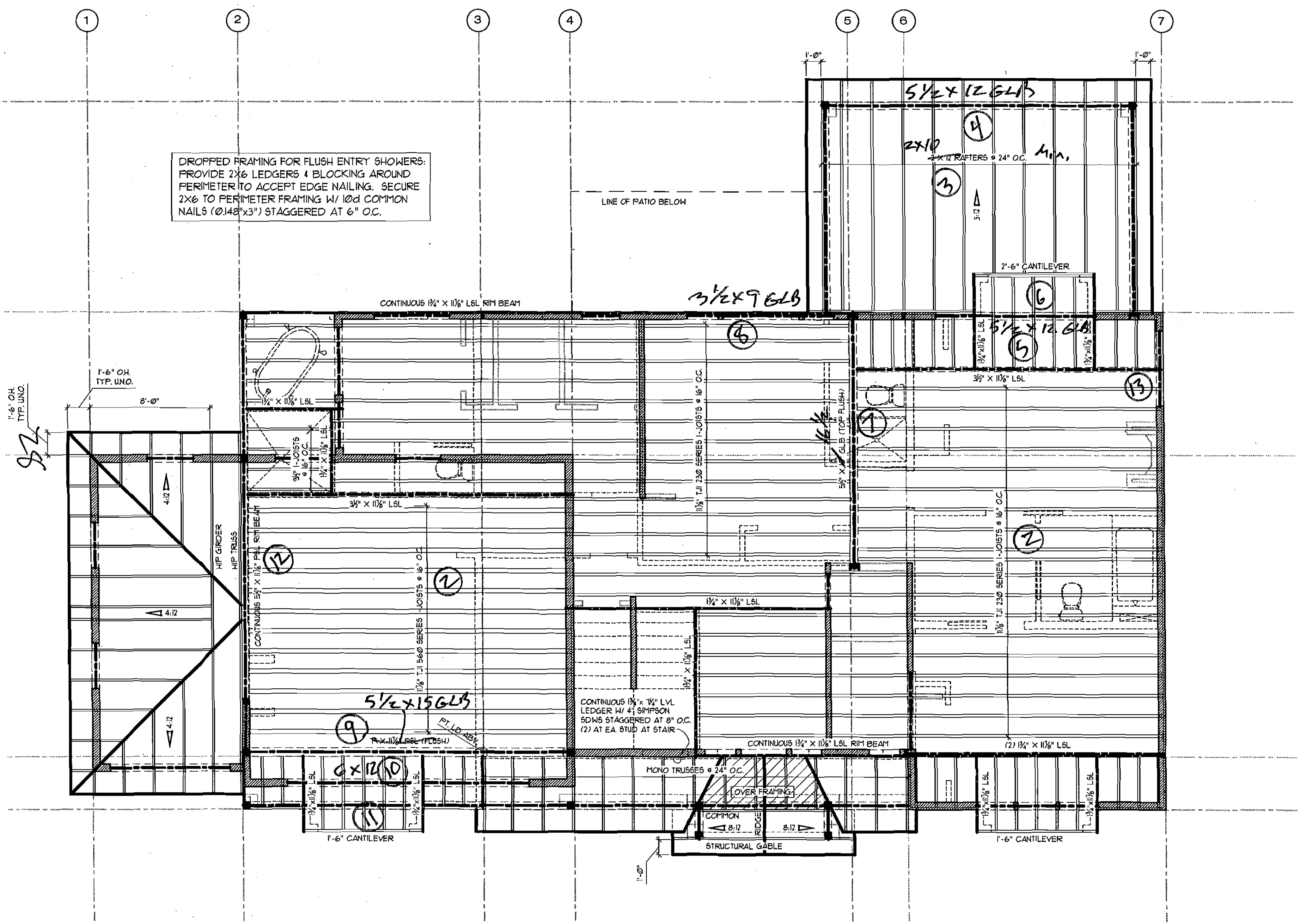
$$\frac{ve\cdot Le - vee\cdot Lee}{33ft} = 111.92\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le - E_{ee}\cdot Lee}{33ft} = 72.65\,ft^{-1}\cdot lb \qquad \frac{ve\cdot Le}{33ft} = 177.76\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le}{33ft} = 205.9\,ft^{-1}\cdot lb$$

Wall Line G:

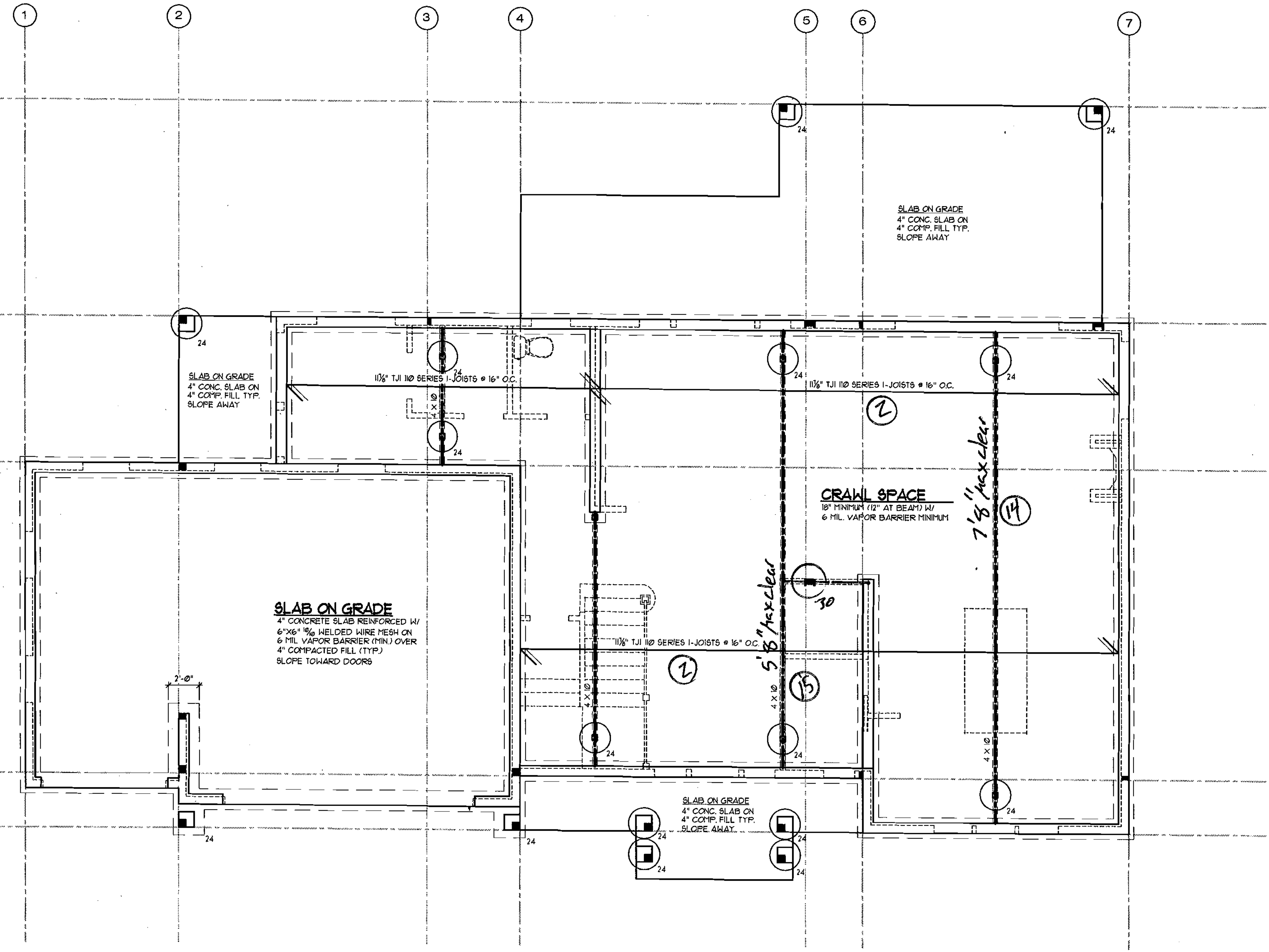
$$\frac{vg\cdot Lg}{21ft} = 28.83\,ft^{-1}\cdot lb \qquad \frac{E_g\cdot Lg}{21ft} = 18.72\,ft^{-1}\cdot lb$$



1
2
3
4
5
6
7
LINE OF ROOF BELOW
1'-6" O.H.
TYP. U.N.O.
GABLE END TRUSS
COMMON TRUSSES @ 24" O.C.
SCISSOR TRUSSES @ 24" O.C.
RIDGE
8:12
OVER FRAMING
GIRDER TRUSS
COMMON TRUSSES @ 24" O.C.
4:33:12
100 PLF DRAG TRUSS
STUBBED TRUSS
COMMON TRUSS
LINE OF PITCH CHANGE
BOX GIRDER
2.5:12
2X12 RAFTERS @ 24" O.C.
HGA10-KT
4 X 8 HEADER AT RAKED WALL
SCISSOR GABLE
MONO TRUSSES @ 24" O.C.
6"
4'-0"
1'-0"
GABLE GIRDER
PRE-MANUF. COMMON TRUSSES @ 24" O.C.
SECURE 2X6 DORMER CRIPPLE WALL BASE PLATES TO GIRDER TRUSS BELOW W/ ¼"x4" SDS SCREWS STAGGERED AT 8" O.C. PROVIDE 2X4 FLAT BLOCKING BETWEEN GIRDER & ADJACENT TRUSS TOP CHORDS @ 24" O.C. W/ A35 EACH END BELOW DORMER WALLS.
5K
5.4K
4x10
4x8
22



1
2
3
4
5
6
7
DROPPED FRAMING FOR FLUSH ENTRY SHOWERS: PROVIDE 2X6 LEDGERS & BLOCKING AROUND PERIMETER TO ACCEPT EDGE NAILING. SECURE 2X6 TO PERIMETER FRAMING W/ 10d COMMON NAILS (0.148"x3") STAGGERED AT 6" O.C.
LINE OF PATIO BELOW
5½x12GLB
2x10
2x12 RAFTERS @ 24" O.C.
Min.
3:12
2'-6" CANTILEVER
1'-0"
CONTINUOUS 1¾" X 11⅞" LSL RIM BEAM
3½x9GLB
3½" X 11⅞" LSL
5½" X 18" GLB (TOP FLUSH)
11⅞" TJI 230 SERIES I-JOISTS @ 16" O.C.
1¾" X 11⅞" LSL
1'-6" O.H. TYP. UNO.
8'-0"
9½" I-JOISTS @ 16" O.C.
HIP GIRDER
HIP TRUSS
4:12
CONTINUOUS 5¼" X 11⅞" PSL RIM BEAM
11⅞" TJI 560 SERIES I-JOISTS @ 16" O.C.
5½x15GLB
PT. LD. ABV.
(FLUSH)
6x12
CONTINUOUS 1¾"x 7¼" LVL LEDGER W/ 4" SIMPSON SDWS STAGGERED AT 8" O.C. (2) AT EA. STUD AT STAIR
MONO TRUSSES @ 24" O.C.
OVER FRAMING
COMMON
RIDGE
8:12
STRUCTURAL GABLE
(2) 1¾" X 11⅞" LSL
1'-6" CANTILEVER



1
2
3
4
5
6
7
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED FILL (TYP.)
SLOPE TOWARD DOORS
2'-0"
CRAWL SPACE
18" MINIMUM (12" AT BEAM) W/
6 MIL. VAPOR BARRIER MINIMUM
11 7/8" TJI 110 SERIES I-JOISTS @ 16" O.C.
4 X 10
24
30
2
14
15
5'8" max clear
7'8" max clear

29

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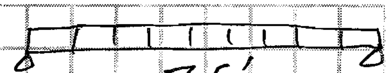
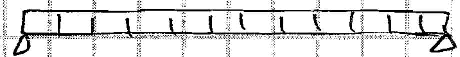
(1) $w_D = 15psf(32'/2) = 240plf$
$w_S = 25psf(32'/2) = 400plf$



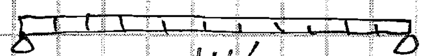
7.5'
4x10
(4x8 @ 6' span)

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



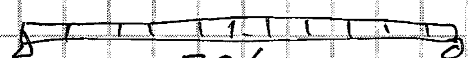
16" o.c.
11 7/8 TJI 560 to 23'8" span
11 7/8 TJI 230 to 19'10" span
11 7/8 TJI 110 to 17'8" span

(3) $w_D = 15psf$
$w_S = 25psf$



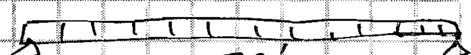
14'
2x10 DF#2 @ 24" o.c.
Minimum

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



20'
5 1/2 x 12 GLB

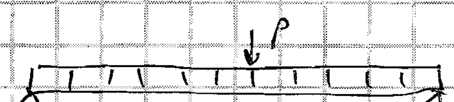
(5) $w_D = 15psf(5'/2) = 37.5plf$
$w_L = 40psf(5'/2) = 100plf$



20'
3 1/2 x 11 7/8 LSL

(6) $w_D = 15psf(9'/2 + 14'/2 + 4'/2) = 202.5plf$
$w_L = 40psf(9'/2) = 180plf$
$w_S = 25psf(14'/2 + 4'/2) = 225plf$

$P = 300^{\#}$ midspan (Door)



10.5'
5 1/2 x 12 GLB

FOR RFA
JOB 9026 61st

DATE 9-30-21
BY [signature]

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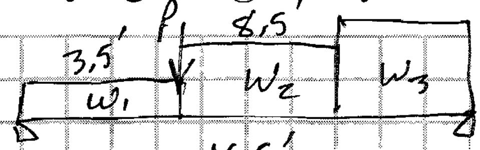
(7) $w_{D_1} = 15psf(14'/2) = 105plf$
$w_{L_1} = 40psf(14'/2) = 280plf$

$w_{D2} = 15psf(34'/2) = 255plf$
$w_{L2} = 40psf(34'/2) = 680plf$

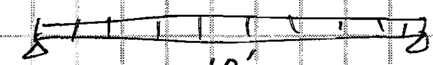
$w_{D3} = 15psf(39'/2) = 292.5plf$
$w_{L3} = 40psf(39'/2) = 780plf$

$P = 375$# DL + 1000# LL from (5)



5½x16½ GLB

(8) $w_D = 15psf(32'/2 + 1') = 255plf$
$w_L = 40plf$
$w_S = 25psf(32'/2) = 400plf$



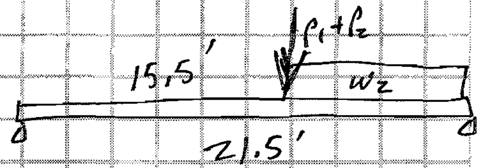
3½x9 GLB

(9) $w_{D_1} = 15psf(4'/2) = 30plf$
$w_{L_1} = 40psf(4'/2) = 80plf$

$w_{D2} = 12psf(9') + 15psf(32'/2) = 348plf$
$w_{S2} = 25psf(32'/2) = 400plf$

$P_1 = 2025$# DL + 2975# SL from Girder Truss

$P_2 = 500$# DL + 475# SL from Rim Beam



5½x15 GLB

FOR RFA
JOB 9026 61st

DATE 9-30-21
BY MM

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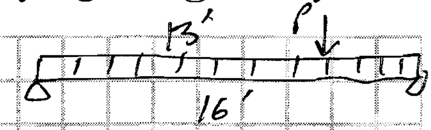
(10) $w_D = 15psf(4'/2) + 12psf(9') + 15psf(2') = 168plf$
$w_L = 40psf(4'/2) = 80plf$
$w_S = 25psf(4'/2) = 50plf$

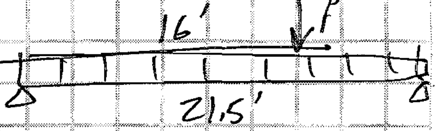
$P = 500^{\#}DL + 475^{\#}SL$ from Rim Beam



6x12

(11) ~~$w_D = 15psf(5'/2 + 2') + 12psf(9') = 175.5plf$~~
~~$w_L = 40psf(5'/2) = 100plf$~~
~~$w_S = 25psf(2') = 50plf$~~

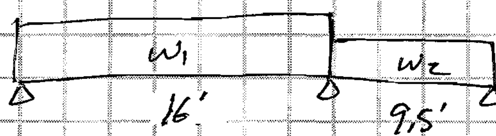
~~$P = 500^{\#}DL + 475^{\#}SL$ from Rim Beam~~



SEE NEXT Page

(12) $w_{D_1} = 15psf(19'/2 + 1' + 22'/2) + 12psf(9') = 430.5plf$
$w_{L_1} = 40psf(22'/2) = 440plf$
$w_{S_1} = 25psf(19'/2 + 1') = 262.5plf$

$w_{D2} = 15psf(19'/2 + 1' + 6'/2) + 12psf(9') = 310.5plf$
$w_{L2} = 40psf(6'/2) = 120plf$
$w_{S2} = 25psf(19'/2) = 237.5plf$

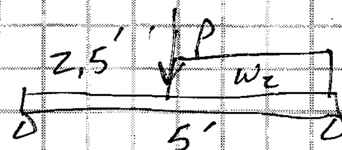


5 1/4 x 11 7/8 PSL

(13) $w_{D_1} = 12psf(9') = 108plf$

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

$P = 375^{\#}DL + 1000^{\#}LL$ from (5)



4x8

FOR RFA
JOB 9026 61st

DATE 9-30-21
BY MM

MYERS ENGINEERING

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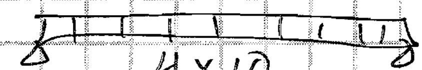
Gig Harbor, WA 98335
(253) 858-3248
Fax (253) 858-3249
myengineer@centurytel.net

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

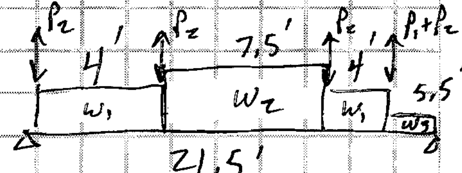


4x10
7'8" clear span

(15) $w_D = 15psf(22'/2 + 18'/2) = 300plf$
$w_L = 40psf(22'/2 + 18'/2) = 800plf$

4x10
5'8" clear span

(11) $w_{D1} = 15psf(2'/2 + 2') + 12psf(9') = 153plf$
$w_{L1} = 40psf(2'/2) = 40plf$
$w_{S1} = 25psf(2') = 50plf$



5½ x 12 GLB

$w_{D2} = 15psf(5'/2 + 2') + 12psf(9') = 175.5plf$
$w_{L2} = 40psf(5'/2) = 100plf$
$w_{S2} = 25psf(2') = 50plf$

$w_{D3} = 15psf(5'/2) = 37.5plf$
$w_{S3} = 25psf(5'/2) = 62.5plf$

$P_1 = 251^{\#} DL + 240^{\#} SL$ from Rim Beam

$P_2 = \pm 920$ WL ± 1860 EL $(\Omega = 3.0)$

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Wood Beam

DESCRIPTION: 1. Upper Header

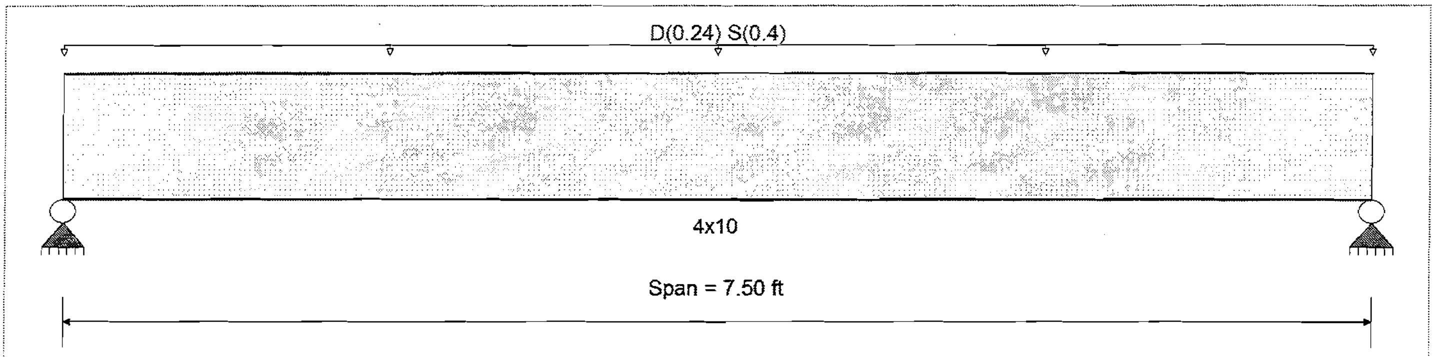
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.240, S = 0.40 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.871**: 1	Maximum Shear Stress Ratio	=	**0.427** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,081.92 psi		=	88.47 psi
	=	1,242.00 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.750 ft	Location of maximum on span	=	6.734 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.078 in	Ratio =	1160 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.124 in	Ratio =	725 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.400	2.400
Overall MINimum	1.500	1.500
D Only	0.900	0.900
+D+L	0.900	0.900
+D+S	2.400	2.400
+D+0.750L	0.900	0.900
+D+0.750L+0.750S	2.025	2.025
+0.60D	0.540	0.540
S Only	1.500	1.500

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Wood Beam

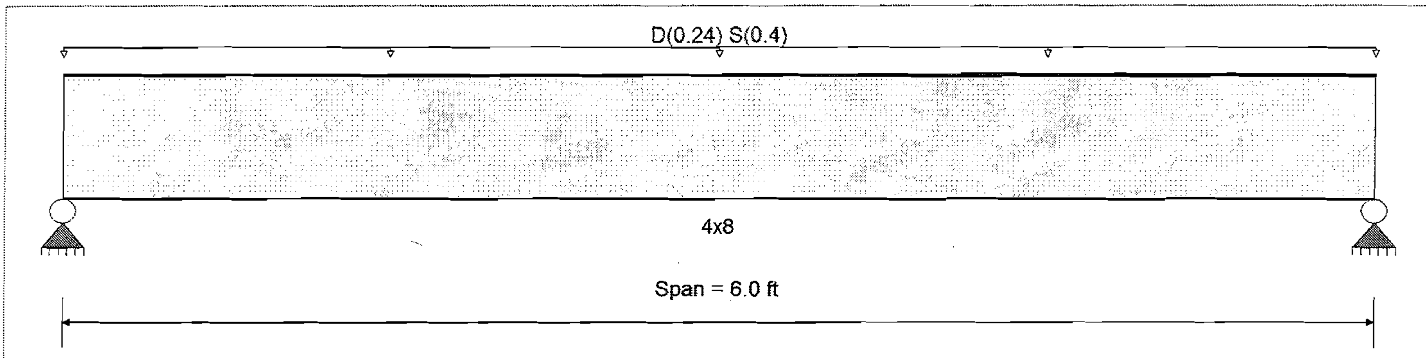
DESCRIPTION: 1a. Upper Header

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.240, S = 0.40 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.838** : 1	Maximum Shear Stress Ratio	=	**0.440** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,127.15 psi		=	91.13 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.066 in	Ratio =	1091 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.106 in	Ratio =	682 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.920	1.920
Overall MINimum	1.200	1.200
D Only	0.720	0.720
+D+L	0.720	0.720
+D+S	1.920	1.920
+D+0.750L	0.720	0.720
+D+0.750L+0.750S	1.620	1.620
+0.60D	0.432	0.432
S Only	1.200	1.200

(2)

L/480 Live Load Deflection

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

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

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

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

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

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

How to Use These Tables

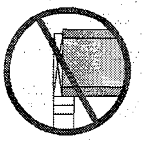
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

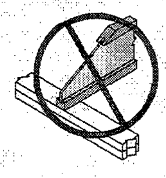
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor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

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

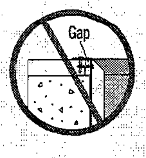
These Conditions Are <u>NOT</u> Permitted:



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



DO NOT bevel cut joist beyond inside face of wall.



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

36

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Wood Beam

File: 9026 SE 61st.ec6
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MYERS ENGINEERING

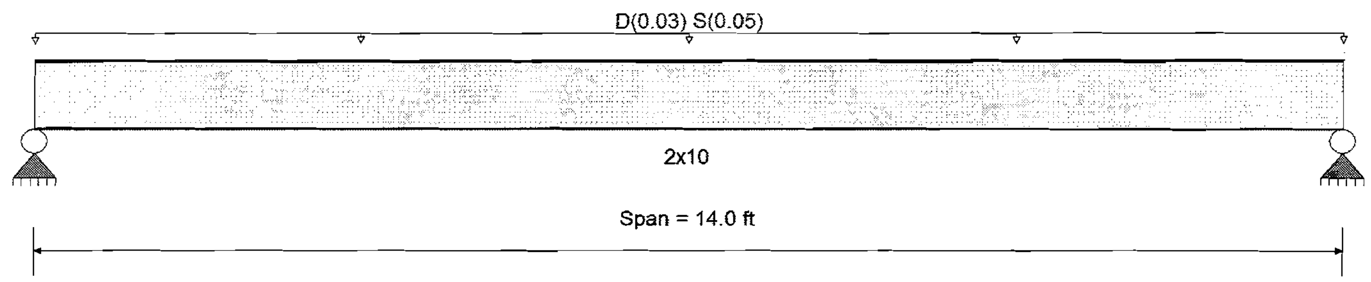
DESCRIPTION: 3. Rafters

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.840** : 1	Maximum Shear Stress Ratio	=	**0.260** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	1,099.55 psi		=	53.91 psi
	=	1,309.28 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.000 ft	Location of maximum on span	=	13.234 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.275 in	Ratio =	611 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.439 in	Ratio =	382 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	0.560	0.560
Overall MINimum	0.350	0.350
D Only	0.210	0.210
+D+L	0.210	0.210
+D+S	0.560	0.560
+D+0.750L	0.210	0.210
+D+0.750L+0.750S	0.473	0.473
+0.60D	0.126	0.126
S Only	0.350	0.350

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Wood Beam

DESCRIPTION: 4. Back Patio Beam

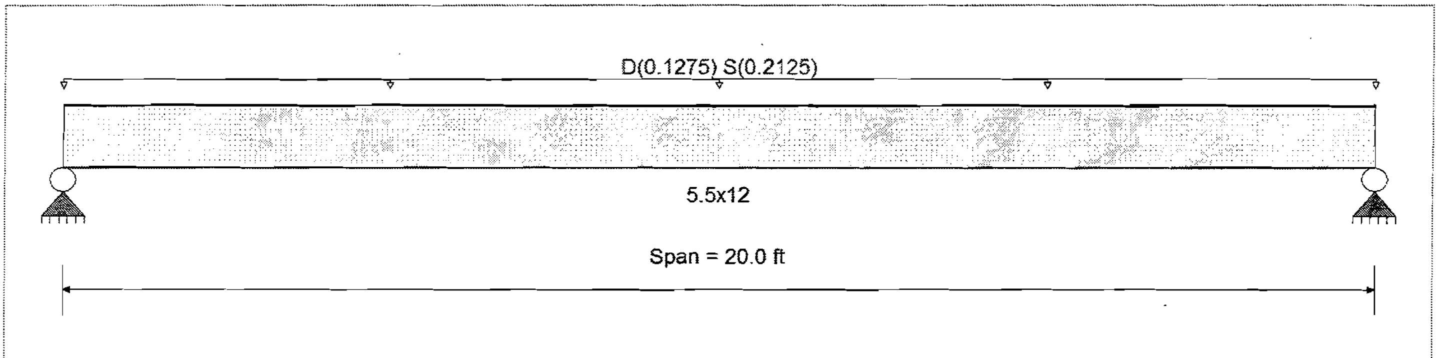
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 8.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.561**: 1	Maximum Shear Stress Ratio	=	**0.230** : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,545.45psi		=	69.94 psi
	=	2,753.98psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.540 in	Ratio =	444 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.864 in	Ratio =	277 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.400	3.400
Overall MINimum	2.125	2.125
D Only	1.275	1.275
+D+L	1.275	1.275
+D+S	3.400	3.400
+D+0.750L	1.275	1.275
+D+0.750L+0.750S	2.869	2.869
+0.60D	0.765	0.765
S Only	2.125	2.125

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Wood Beam

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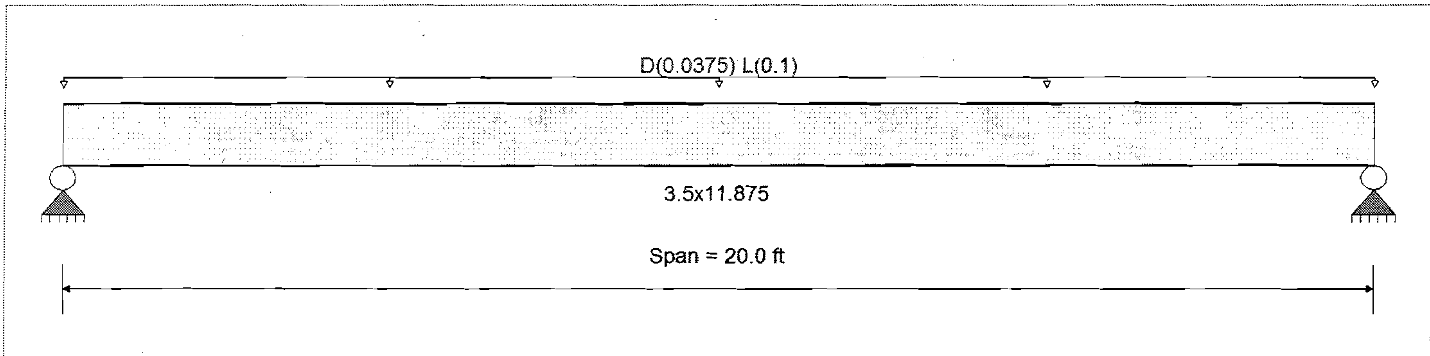
DESCRIPTION: 5. Floor beam at Kid's Bed Rm

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.03750, L = 0.10 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.431**: 1	Maximum Shear Stress Ratio	=	**0.145** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
	=	1,002.93psi		=	44.92 psi
	=	2,325.00psi		=	310.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	10.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.478 in	Ratio =	501 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.658 in	Ratio =	364 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.375	1.375
Overall MINimum	1.000	1.000
D Only	0.375	0.375
+D+L	1.375	1.375
+D+S	0.375	0.375
+D+0.750L	1.125	1.125
+D+0.750L+0.750S	1.125	1.125
+0.60D	0.225	0.225
L Only	1.000	1.000
S Only		

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Wood Beam

DESCRIPTION: 6. Header at Great Rm accordion door

CODE REFERENCES

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

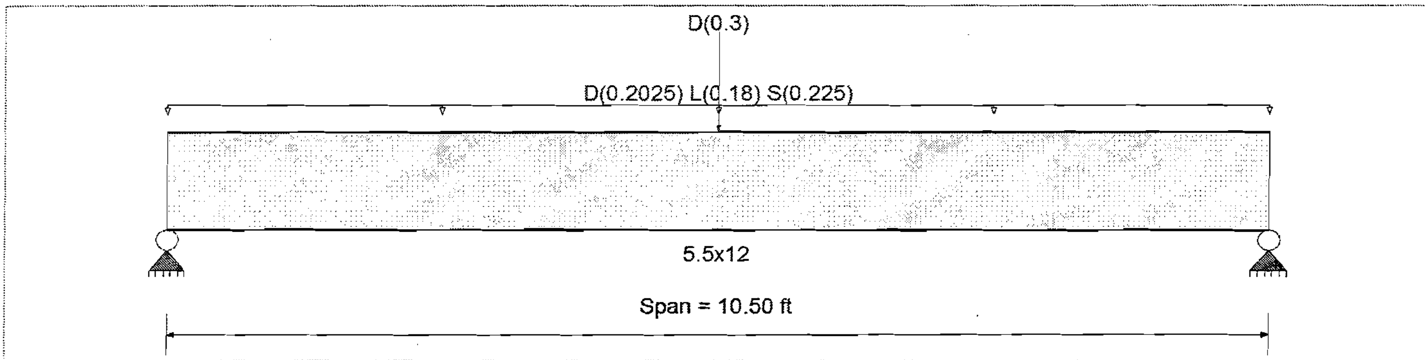
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

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

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

Fb +	2400 psi	*E : Modulus of Elasticity*	
Fb -	1850 psi	Ebend- xx	1800 ksi
Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Fc - Perp	650 psi	Ebend- yy	1600 ksi
Fv	265 psi	Eminbend - yy	850 ksi
Ft	1100 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.2025, L = 0.180, S = 0.2250 , Tributary Width = 1.0 ft
Point Load : D = 0.30 k @ 5.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.256** 1	Maximum Shear Stress Ratio	=	**0.172** : 1
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	705.84 psi		=	52.35 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	5.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.043 in	Ratio =	2902 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.107 in	Ratio =	1183 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.808	2.808
Overall MINimum	1.181	1.181
D Only	1.213	1.213
+D+L	2.158	2.158
+D+S	2.394	2.394
+D+0.750L	1.922	1.922
+D+0.750L+0.750S	2.808	2.808
+0.60D	0.728	0.728
L Only	0.945	0.945
S Only	1.181	1.181

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Wood Beam

DESCRIPTION: 7. Beam over Great Rm/Dining Rm

CODE REFERENCES

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

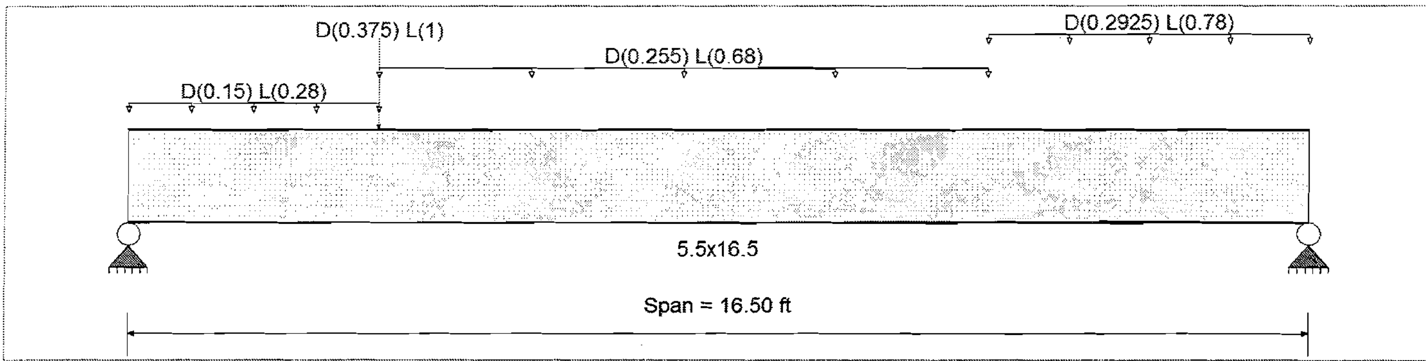
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

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

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

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.150, L = 0.280 k/ft, Extent = 0.0 -->> 3.50 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.2550, L = 0.680 k/ft, Extent = 3.50 -->> 12.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.2925, L = 0.780 k/ft, Extent = 12.0 -->> 16.50 ft, Tributary Width = 1.0 ft
Point Load : D = 0.3750, L = 1.0 k @ 3.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.679: 1**	Maximum Shear Stress Ratio	=	**0.432 : 1**
Section used for this span		**5.5x16.5**	Section used for this span		**5.5x16.5**
	=	1,604.81 psi		=	114.57 psi
	=	2,364.76 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.250 ft	Location of maximum on span	=	15.175 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.323 in	Ratio =	613 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.446 in	Ratio =	444 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	7.301	8.352
Overall MINimum	5.208	6.062
D Only	2.094	2.290
+D+L	7.301	8.352
+D+S	2.094	2.290
+D+0.750L	5.999	6.837
+D+0.750L+0.750S	5.999	6.837
+0.60D	1.256	1.374
L Only	5.208	6.062
S Only		

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Wood Beam

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DESCRIPTION: 8. Dining Rm header

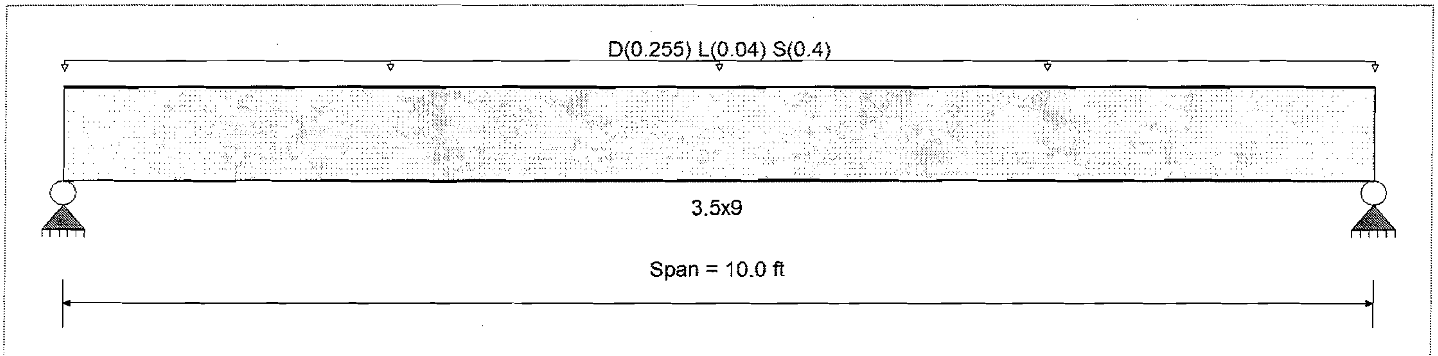
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.2550, L = 0.040, S = 0.40 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.753: 1**	Maximum Shear Stress Ratio	=	**0.437 : 1**
Section used for this span		**3.5x9**	Section used for this span		**3.5x9**
	=	2,079.37 psi		=	133.19 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.237 in	Ratio =	507 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.387 in	Ratio =	309 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.275	3.275
Overall MINimum	2.000	2.000
D Only	1.275	1.275
+D+L	1.475	1.475
+D+S	3.275	3.275
+D+0.750L	1.425	1.425
+D+0.750L+0.750S	2.925	2.925
+0.60D	0.765	0.765
L Only	0.200	0.200
S Only	2.000	2.000

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Wood Beam

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DESCRIPTION: 9. Floor Beam over Garage

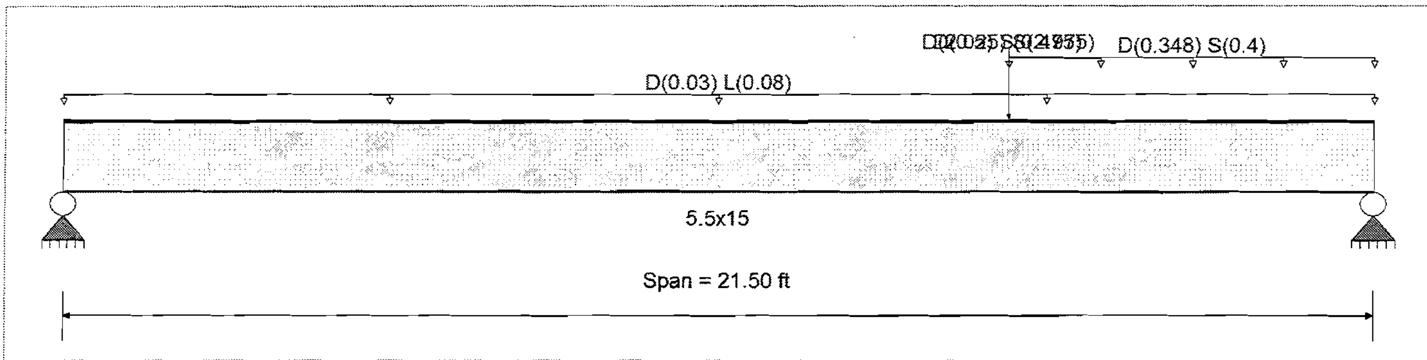
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.030, L = 0.080 , Tributary Width = 1.0 ft
Uniform Load : D = 0.3480, S = 0.40 k/ft, Extent = 15.50 -->> 21.50 ft, Tributary Width = 1.0 ft
Point Load : D = 2.025, S = 2.975 k @ 15.50 ft
Point Load : D = 0.50, S = 0.4750 k @ 15.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.802** 1	Maximum Shear Stress Ratio	=	**0.452** : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	2,144.39psi		=	137.75 psi
	=	2,673.80psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	15.458ft	Location of maximum on span	=	20.323 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.464 in	Ratio =	555 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.873 in	Ratio =	295 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.937	8.492
Overall MINimum	1.298	4.552
D Only	1.319	3.940
+D+L	2.179	4.800
+D+S	2.616	8.492
+D+0.750L	1.964	4.585
+D+0.750L+0.750S	2.937	7.999
+0.60D	0.791	2.364
L Only	0.860	0.860
S Only	1.298	4.552

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Wood Beam

DESCRIPTION: 10. Garage Door Header

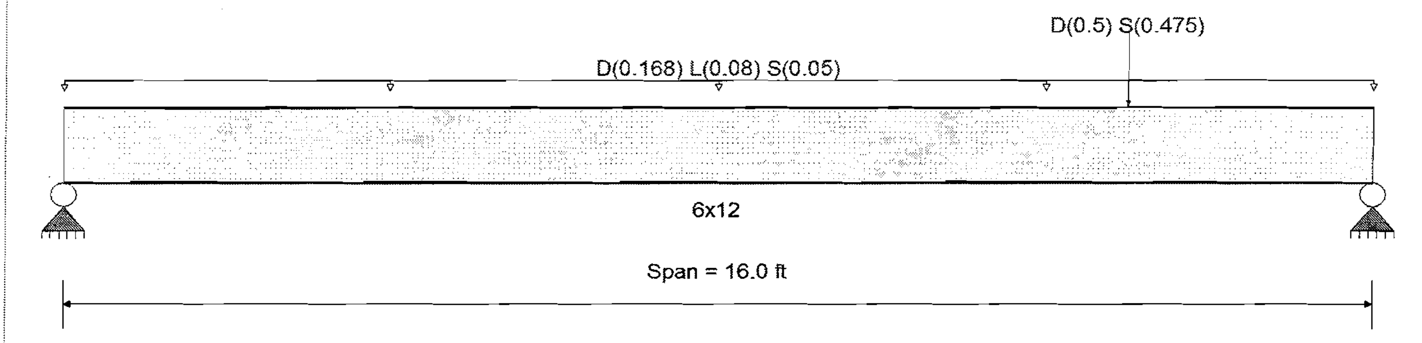
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1680, L = 0.080, S = 0.050 , Tributary Width = 1.0 ft
Point Load : D = 0.50, S = 0.4750 k @ 13.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.985 : 1**	Maximum Shear Stress Ratio	=	**0.312 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	861.54 psi		=	60.99 psi
	=	875.00 psi		=	195.50 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.350 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.131 in	Ratio =	1466 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.510 in	Ratio =	376 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.285	2.820
Overall MINimum	0.489	0.786
D Only	1.438	1.750
+D+L	2.078	2.390
+D+S	1.927	2.536
+D+0.750L	1.918	2.230
+D+0.750L+0.750S	2.285	2.820
+0.60D	0.863	1.050
L Only	0.640	0.640
S Only	0.489	0.786

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Wood Beam

DESCRIPTION: 11. Beam in front of Garage

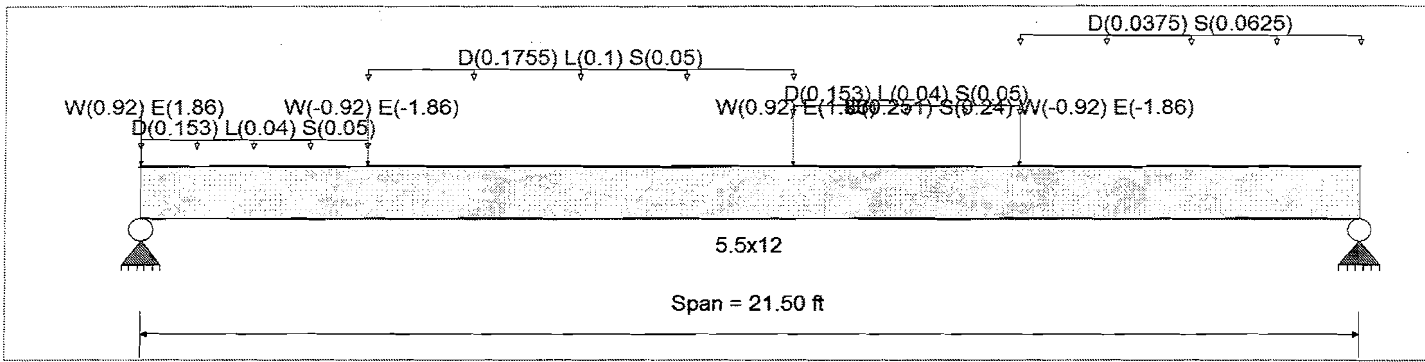
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.1530, L = 0.040, S = 0.050 k/ft, Extent = 0.0 -->> 4.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.1530, L = 0.040, S = 0.050 k/ft, Extent = 11.50 -->> 15.50 ft, Tributary Width = 1.0 ft
- Point Load : W = 0.920, E = 1.860 k @ 0.0 ft
- Point Load : W = -0.920, E = -1.860 k @ 4.0 ft
- Point Load : W = 0.920, E = 1.860 k @ 11.50 ft
- Point Load : D = 0.2510, S = 0.240, W = -0.920, E = -1.860 k @ 15.50 ft
- Uniform Load : D = 0.1755, L = 0.10, S = 0.050 k/ft, Extent = 4.0 -->> 11.50 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.03750, S = 0.06250 k/ft, Extent = 15.50 -->> 21.50 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.496: 1**	Maximum Shear Stress Ratio	=	**0.243 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	1,178.96 psi		=	102.86 psi
	=	2,377.51 psi		=	424.00 psi
Load Combination		+D+L	Load Combination		+1.122D+0.750L+0.750S-1.575E
Location of maximum on span	=	9.965 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.223 in	Ratio =	1158 >=360
Max Upward Transient Deflection	-0.138 in	Ratio =	1869 >=360
Max Downward Total Deflection	0.928 in	Ratio =	277 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	3.064	2.524
Overall MINimum	-0.692	0.692
D Only	1.726	1.290
+D+L	2.410	1.676
+D+S	2.341	2.065
+D+0.750L	2.239	1.580
+D+0.750L+0.750S	2.700	2.161

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DESCRIPTION: 11. Beam in front of Garage

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.60W	1.931	1.085
+D-0.60W	1.521	1.496
+D+0.70E	2.210	0.806
+D-0.70E	1.242	1.775
+D+0.750L+0.450W	2.393	1.425
+D+0.750L-0.450W	2.085	1.734
+D+0.750L+0.750S+0.450W	2.855	2.007
+D+0.750L+0.750S-0.450W	2.546	2.315
+D+0.750L+0.750S+0.5250E	3.064	1.797
+D+0.750L+0.750S-0.5250E	2.337	2.524
+0.60D+0.60W	1.241	0.569
+0.60D-0.60W	0.830	0.980
+0.60D+0.70E	1.520	0.290
+0.60D-0.70E	0.551	1.259
L Only	0.684	0.386
S Only	0.615	0.775
W Only	0.342	-0.342
-W	-0.342	0.342
E Only	0.692	-0.692
E Only * -1.0	-0.692	0.692

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DESCRIPTION: 12. Rim Beam at Grid 2

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :IBC 2018

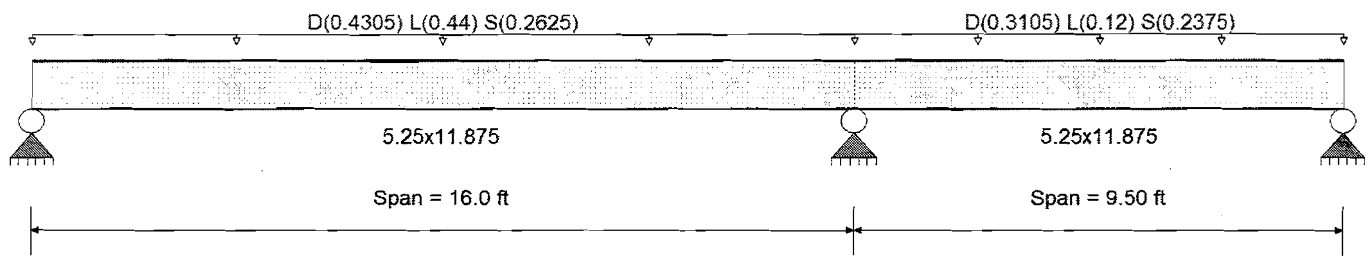
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

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

Fb +	2,900.0	psi
Fb -	2,900.0	psi
Fc - Prll	2,900.0	psi
Fc - Perp	750.0	psi
Fv	290.0	psi
Ft	2,025.0	psi

E : Modulus of Elasticity
Ebend- xx 2,000.0ksi
Eminbend - xx 1,016.54ksi

Density 45.070pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.4305, L = 0.440, S = 0.2625 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.3105, L = 0.120, S = 0.2375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.647**: 1	Maximum Shear Stress Ratio	=	**0.607** : 1
Section used for this span		**5.25x11.875**	Section used for this span		**5.25x11.875**
	=	1,875.79psi		=	175.97 psi
	=	2,900.00psi		=	290.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	16.000ft	Location of maximum on span	=	15.017 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.281 in	Ratio =	683 >=480
Max Upward Transient Deflection	-0.061 in	Ratio =	1862 >=480
Max Downward Total Deflection	0.573 in	Ratio =	334 >=240
Max Upward Total Deflection	-0.093 in	Ratio =	1231 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	6.329	14.040	1.167
Overall MINimum	1.708	4.280	0.468
+D+H	2.822	6.588	0.428
+D+L+H, LL Comb Run (*L)	2.791	7.243	0.945
+D+L+H, LL Comb Run (L*)	5.790	11.590	-0.502
+D+L+H, LL Comb Run (LL)	5.759	12.245	0.015
+D+Lr+H, LL Comb Run (*L)	2.822	6.588	0.428
+D+Lr+H, LL Comb Run (L*)	2.822	6.588	0.428
+D+Lr+H, LL Comb Run (LL)	2.822	6.588	0.428
+D+S+H	4.530	10.868	0.896
+D+0.750Lr+0.750L+H, LL Comb Run (*	2.799	7.079	0.815
+D+0.750Lr+0.750L+H, LL Comb Run (L	5.048	10.339	-0.270

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 12. Rim Beam at Grid 2

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750Lr+0.750L+H, LL Comb Run (L	5.024	10.830	0.118
+D+0.750L+0.750S+H, LL Comb Run (*L	4.080	10.289	1.167
+D+0.750L+0.750S+H, LL Comb Run (L*	6.329	13.549	0.081
+D+0.750L+0.750S+H, LL Comb Run (LL	6.306	14.040	0.469
+D+0.60W+H	2.822	6.588	0.428
+D-0.60W+H	2.822	6.588	0.428
+D+0.70E+H	2.822	6.588	0.428
+D-0.70E+H	2.822	6.588	0.428
+D+0.750Lr+0.750L+0.450W+H, LL Comb	2.799	7.079	0.815
+D+0.750Lr+0.750L+0.450W+H, LL Comb	5.048	10.339	-0.270
+D+0.750Lr+0.750L+0.450W+H, LL Comb	5.024	10.830	0.118
+D+0.750Lr+0.750L-0.450W+H, LL Comb	2.799	7.079	0.815
+D+0.750Lr+0.750L-0.450W+H, LL Comb	5.048	10.339	-0.270
+D+0.750Lr+0.750L-0.450W+H, LL Comb	5.024	10.830	0.118
+D+0.750L+0.750S+0.450W+H, LL Comb	4.080	10.289	1.167
+D+0.750L+0.750S+0.450W+H, LL Comb	6.329	13.549	0.081
+D+0.750L+0.750S+0.450W+H, LL Comb	6.306	14.040	0.469
+D+0.750L+0.750S-0.450W+H, LL Comb	4.080	10.289	1.167
+D+0.750L+0.750S-0.450W+H, LL Comb	6.329	13.549	0.081
+D+0.750L+0.750S-0.450W+H, LL Comb	6.306	14.040	0.469
+D+0.750L+0.750S+0.5250E+H, LL Comb	4.080	10.289	1.167
+D+0.750L+0.750S+0.5250E+H, LL Comb	6.329	13.549	0.081
+D+0.750L+0.750S+0.5250E+H, LL Comb	6.306	14.040	0.469
+D+0.750L+0.750S-0.5250E+H, LL Comb	4.080	10.289	1.167
+D+0.750L+0.750S-0.5250E+H, LL Comb	6.329	13.549	0.081
+D+0.750L+0.750S-0.5250E+H, LL Comb	6.306	14.040	0.469
+0.60D+0.60W+0.60H	1.693	3.953	0.257
+0.60D-0.60W+0.60H	1.693	3.953	0.257
+0.60D+0.70E+0.60H	1.693	3.953	0.257
+0.60D-0.70E+0.60H	1.693	3.953	0.257
D Only	2.822	6.588	0.428
L Only, LL Comb Run (*L)	-0.032	0.655	0.517
L Only, LL Comb Run (L*)	2.968	5.002	-0.930
L Only, LL Comb Run (LL)	2.936	5.657	-0.413
S Only	1.708	4.280	0.468
H Only			

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Wood Beam

DESCRIPTION: 13. Header at Window Seat in Great Rm

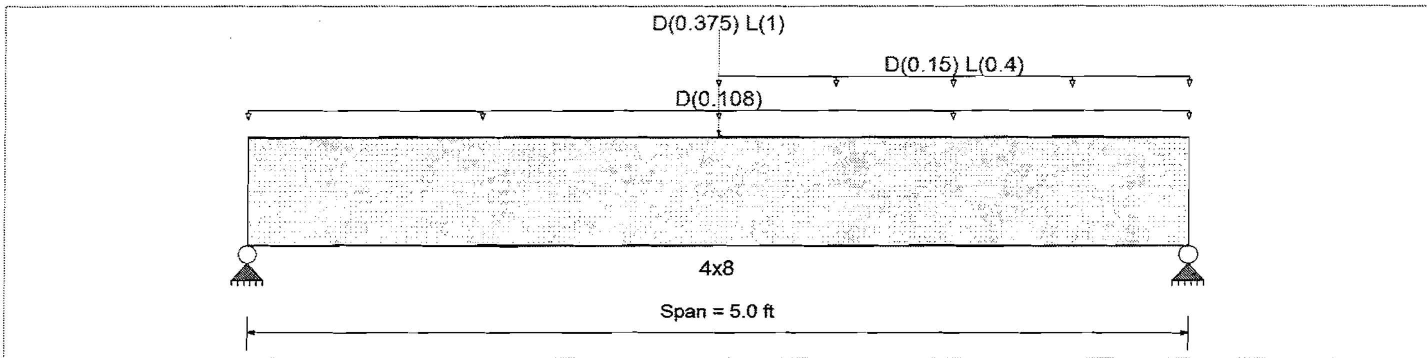
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1080 , Tributary Width = 1.0 ft
Uniform Load : D = 0.150, L = 0.40 k/ft, Extent = 2.50 -->> 5.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.3750, L = 1.0 k @ 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.975 : 1**	Maximum Shear Stress Ratio	=	**0.523 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,141.09 psi		=	94.14 psi
	=	1,170.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.500 ft	Location of maximum on span	=	4.398 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.041 in	Ratio =	1449 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.065 in	Ratio =	916 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.301	1.989
Overall MINimum	0.750	1.250
D Only	0.551	0.739
+D+L	1.301	1.989
+D+S	0.551	0.739
+D+0.750L	1.114	1.676
+D+0.750L+0.750S	1.114	1.676
+0.60D	0.331	0.443
L Only	0.750	1.250
S Only		

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Wood Beam

DESCRIPTION: 14. Crawl beam NOT at brg wall

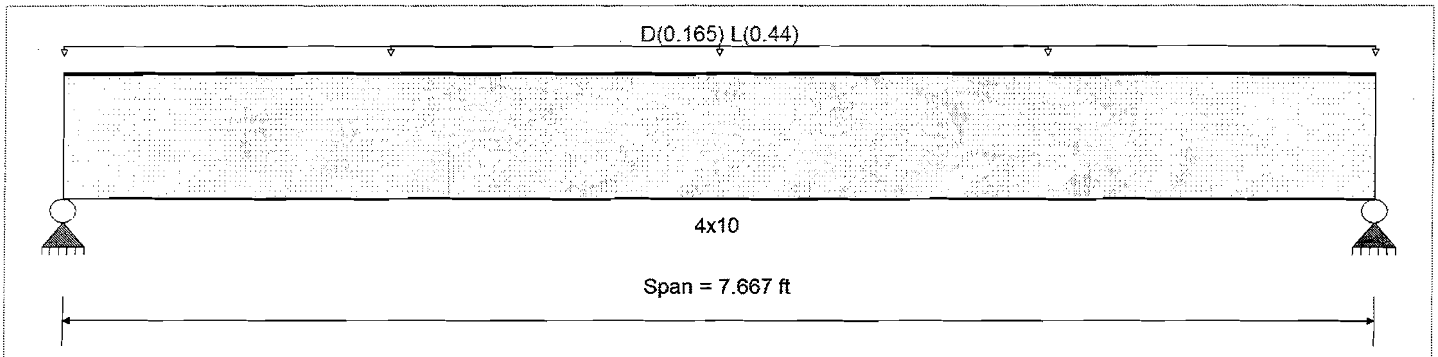
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1650, L = 0.440 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.990 : 1**	Maximum Shear Stress Ratio	=	**0.479 : 1**
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,068.80 psi		=	86.28 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.834 ft	Location of maximum on span	=	6.911 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	987 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.128 in	Ratio =	718 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.319	2.319
Overall MINimum	1.687	1.687
D Only	0.633	0.633
+D+L	2.319	2.319
+D+S	0.633	0.633
+D+0.750L	1.898	1.898
+D+0.750L+0.750S	1.898	1.898
+0.60D	0.380	0.380
L Only	1.687	1.687
S Only		

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Wood Beam

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MYERS ENGINEERING

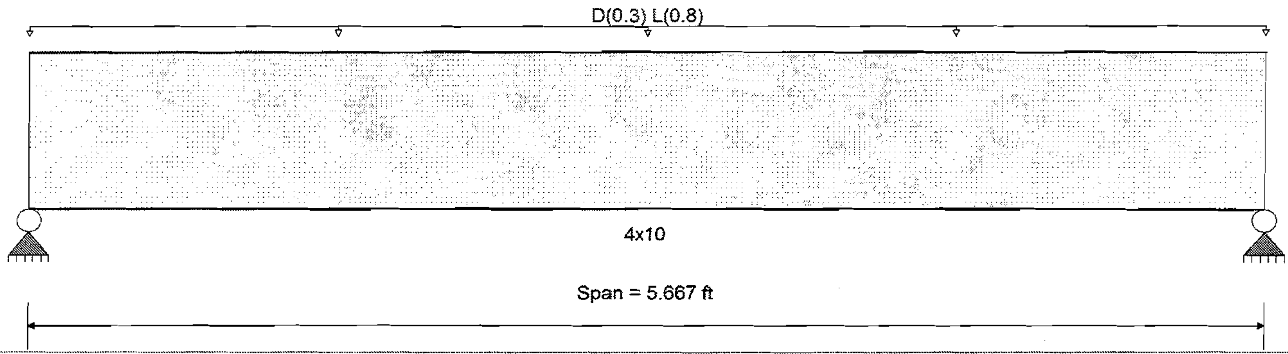
DESCRIPTION: 15. Crawl beam at brg wall

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.30, L = 0.80 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.983** 1	Maximum Shear Stress Ratio	=	**0.586** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,061.67 psi		=	105.41 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.834 ft	Location of maximum on span	=	4.902 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.051 in	Ratio =	1345 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.070 in	Ratio =	978 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.117	3.117
Overall MINimum	2.267	2.267
D Only	0.850	0.850
+D+L	3.117	3.117
+D+S	0.850	0.850
+D+0.750L	2.550	2.550
+D+0.750L+0.750S	2.550	2.550
+0.60D	0.510	0.510
L Only	2.267	2.267
S Only		

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PROJECT : **9026 SE 61st ST**

Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

Maximum Load For 4x4 HF#2 Treated Post

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

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

$E' := 1235000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

4x4 Treated Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

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

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

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

$C_p = 0.6$

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