MYERS ENGINEERING

Lateral & Gravity Calculations

Mark Myers
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DIGITAL PDF SIGNATURE FOR PERMIT SUBMITTAL.

Project: RKK Construction
2434 73rd Avenue Southeast
Mercer Island, WA

July 1, 2025

2021 INTERNATIONAL BUILDING CODE
100 MPH BASIC WIND, EXPOSURE B, K_{zt} = 1.32
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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PROJECT : **2434 73rd AVE SE**

DESIGN LOADS:

ROOF DEAD LOADS	15 PSF Total
ROOF LIVE LOADS	25 PSF (Snow)
FLOOR DEAD LOADS	15 PSF Total
DECK DEAD LOADS	25 PSF Total
FLOOR LIVE LOADS	40 PSF (Reducible)
DECK LIVE LOADS	60 PSF
STAIR LIVE LOADS	100 PSF

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

WOODS :

WOOD TYPE:

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

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

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

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

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD
F_b=1,700 PSI, F_v=400 PSI, F_c (Perp) =680 PSI, E=1,300,000 PSI.

TRUSSES:

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

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

ENGINEERED I-JOISTS

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

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LATERAL ANALYSIS :

BASED ON 2021 INTERNATIONAL BUILDING CODE (IBC)

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

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

SEISMIC DESIGN:

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

Seismic Design Data:

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

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

$S_s := 1.39$ $S_1 := 0.49$ $S_{ms} := 1.39$ $S_{m1} := 0.98$

Equation 11.4-3 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.93$ Equation 11.4-4 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.65$

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

Plan Area for Each Level:

$A_1 := 1.12\ (4020) \cdot ft^2$ $A_{2a} := 2155 \cdot ft^2$ $A_{2b} := (540) \cdot ft^2$

(Upper Roof) (Tributary Floor) (Deck)

Plan Perimeter for Each Level:

$P_1 := 2 \cdot (54 \cdot ft) + 2 \cdot (75 \cdot ft)$ $P_2 := 2 \cdot (30 \cdot ft) + 2 \cdot (73.5 \cdot ft)$

(Upper Floor) (Main Floor)

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

Weight of Structure at Each Level:

Story Weight at Upper Floor:

$w_1 := 15 \cdot psf \cdot A_1 + 12 \cdot psf \cdot 5.5 \cdot ft \cdot P_1$

Story Weight at Main Floor:

$w_2 := 15 \cdot psf \cdot A_{2a} + 25\ psf \cdot A_{2b} + 12 \cdot psf \cdot (5 \cdot ft \cdot P_1 + 4.5 \cdot ft \cdot P_2)$

$W := w_1 + w_2 = 157047.00\ lb$

Roof Adjustment:

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

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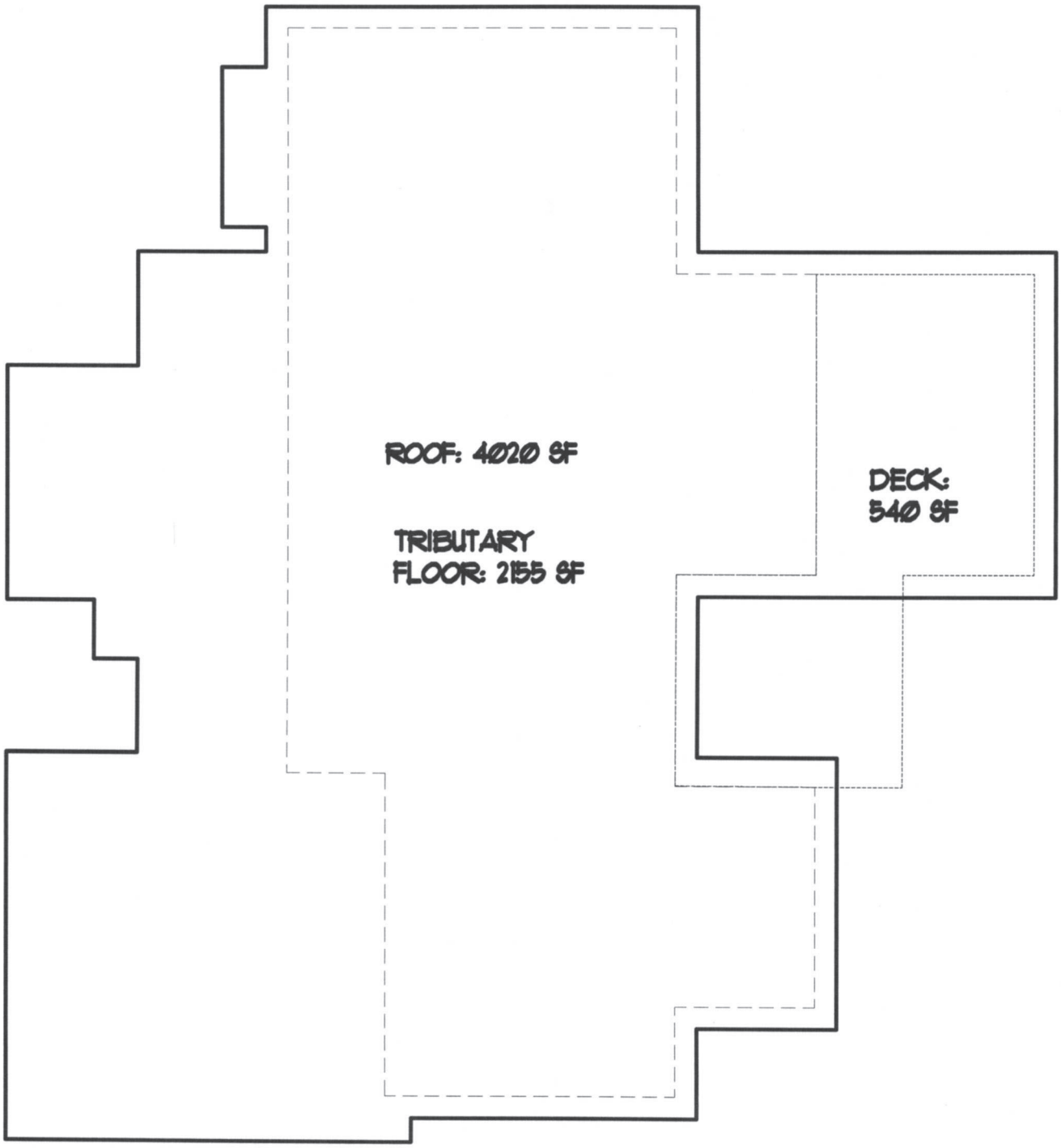
ROOF: 4020 SF
TRIBUTARY
FLOOR: 2155 SF
DECK:
540 SF

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Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $h_n := 25$ (Structural Height per ASCE 7-16 Sect. 11.2)

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

T_a is less than T_L, therefore Cs need not exceed: $\frac{S_{D1}}{\left(\frac{R}{I_e}\right) \cdot T_a} = 0.45$ (ASCE 7-16 Eq. 12.8-3)

C_s shall not be less than: $0.044 \cdot S_{DS} \cdot I_e = 0.04$ (ASCE 7-16 Eq. 12.8-5)

$$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.14$$ (ASCE 7-16 Eq. 12.8-2)

Total Base Shear: $V_E := C_s \cdot W = 22389.26\ \textbf{lb}$

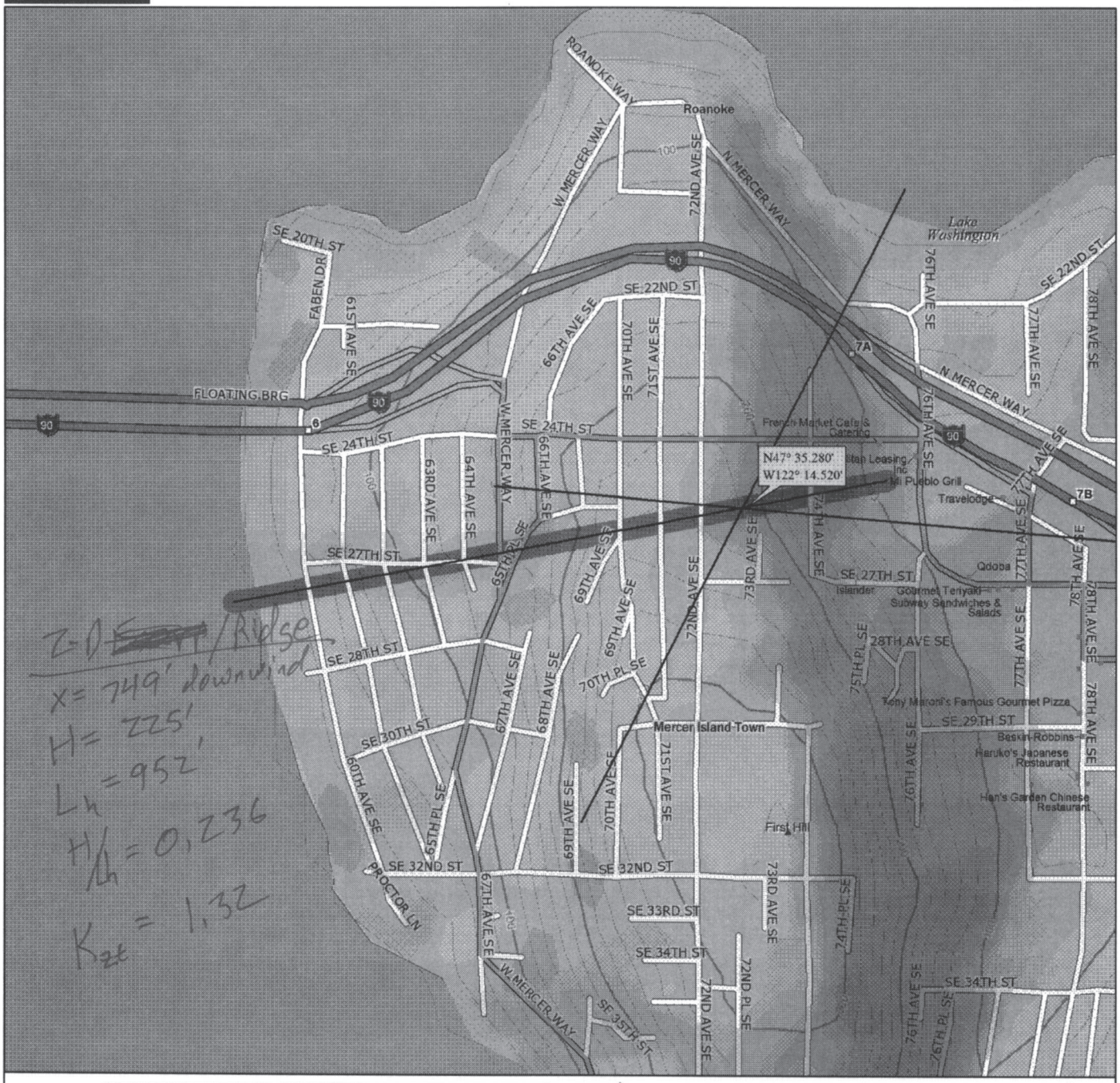
Vertical Shear distribution at each level per ASCE 7-16 Eq. 12.8-12:

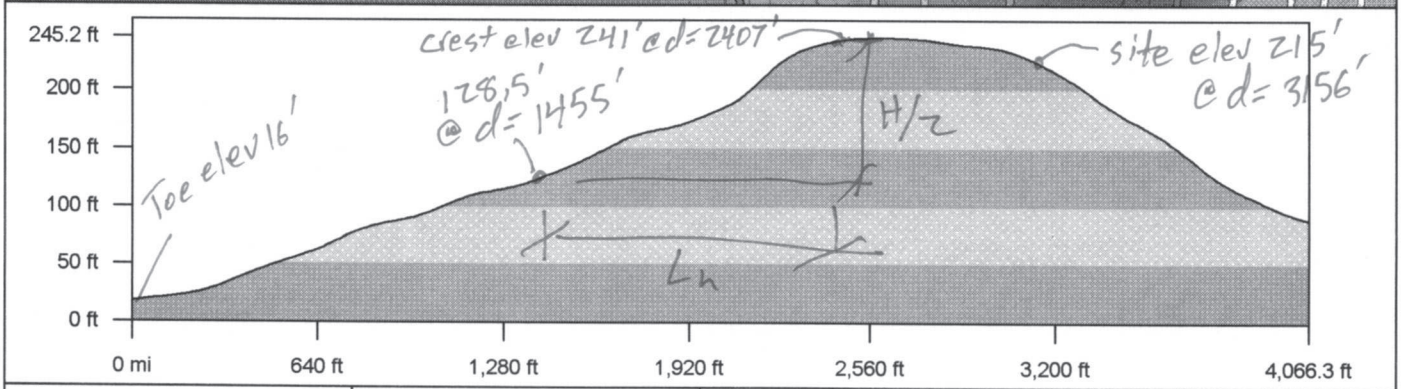
For structures having a period of 0.5 sec or less: $k := 1$

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

$C_{v1} := \frac{\langle w_1 \cdot h_1 \rangle}{\langle w_1 \cdot h_1 + w_2 \cdot h_2 \rangle} = 0.70$ $F_1 := C_{v1} \cdot V_E = 15672.51\ \textbf{lb}$ Story Shear at Upper Floor

$C_{v2} := \frac{\langle w_2 \cdot h_2 \rangle}{\langle w_1 \cdot h_1 + w_2 \cdot h_2 \rangle} = 0.30$ $F_2 := C_{v2} \cdot V_E = 6716.75\ \textbf{lb}$ Story Shear at Main Floor



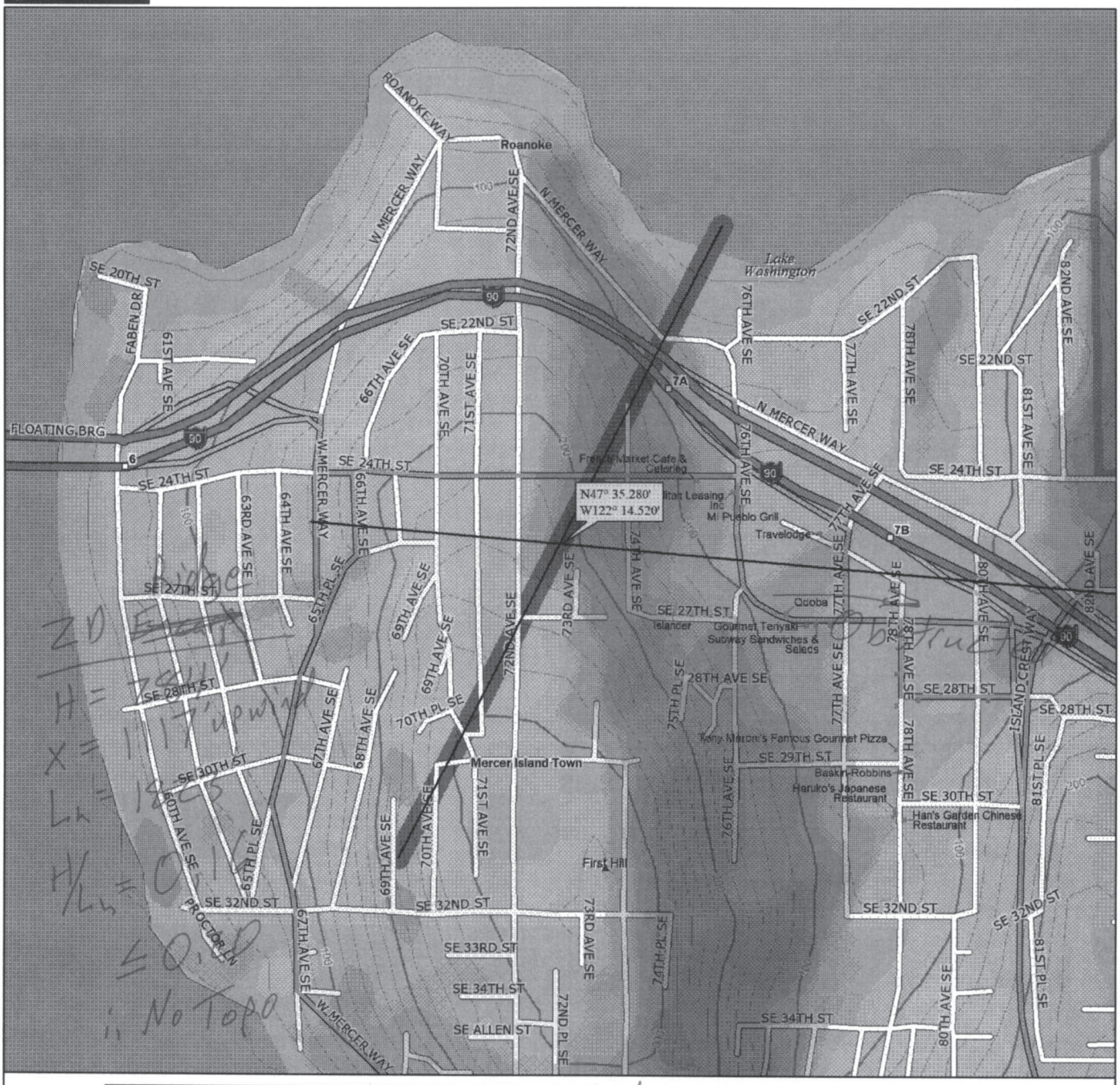


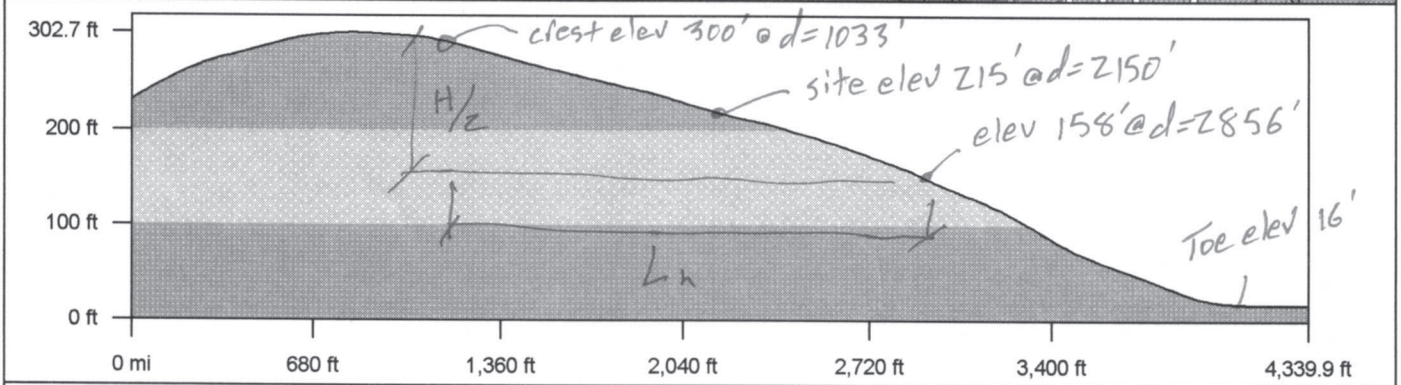
Lin Dist: 4,042.7 ft	Terr Dist: 4,066.3 ft	Elev Gain: 70.8 ft	Avg Grade: 9
Climb Elev: 227.4 ft	Desc Elev: 156.6 ft	Max. Elev: 245.2 ft	Min. Elev: 17.8 ft
Climb Dist: 2,578.3 ft	Desc Dist: 1,488.1 ft		

TN

Scale 1 : 11,200

1" = 933.3 ft Data Zoom 14-2





Lin Dist: 4,321.3 ft	Terr Dist: 4,339.9 ft	Elev Gain: -215.3 ft	Avg Grade: 8
Climb Elev: 71.0 ft	Desc Elev: 286.4 ft	Max. Elev: 302.7 ft	Min. Elev: 16.4 ft
Climb Dist: 912.9 ft	Desc Dist: 3,427.0 ft		

Scale 1 : 11,200
1" = 933.3 ft
Data Zoom 14-2

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WIND DESIGN

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

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

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

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

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

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

$x := 749 \cdot ft$ $H := 225 \cdot ft$ $L_h := 952 \cdot ft$ $z := h$ $\gamma := 3$ $\mu := 1.5$

$K_1 := 1.45 \cdot \left(\frac{H}{L_h}\right) = 0.34$ $K_2 := \left(1 - \frac{x}{\mu \cdot L_h}\right) = 0.48$ $K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.92$ $K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1.32$

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

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

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

Velocity Pressure Exposure Coefficient (Table 26.10-1):

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

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

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

External Pressure Coefficients w/ Roof Pitch = 5/12 (22.6 degrees) Front to Back & 7/12 (30 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 54 \cdot ft$ $B_{fb} := 75 \cdot ft$ $\frac{L_{fb}}{B_{fb}} = 0.72$ $\frac{h}{L_{fb}} = 0.46$

$C_{pf1} := 0.8$ Windward Wall

$C_{pf2} := 0.02$ Windward Roof

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

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

Side to Side:

$L_{ss} := 75 \cdot ft$ $B_{ss} := 54 \cdot ft$ $\frac{L_{ss}}{B_{ss}} = 1.39$ $\frac{h}{L_{ss}} = 0.33$

$C_{ps1} := 0.8$ Windward Wall

$C_{ps2} := 0.27$ Windward Roof

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

$C_{ps4} := -0.42$ Leeward Wall

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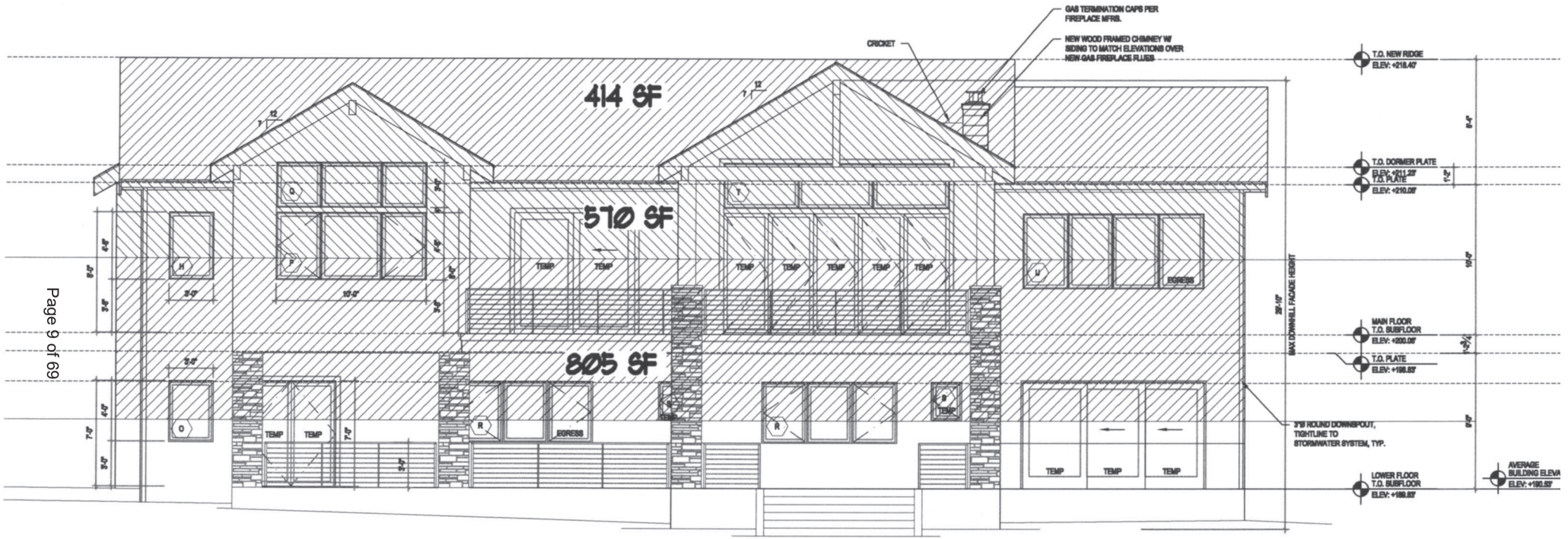
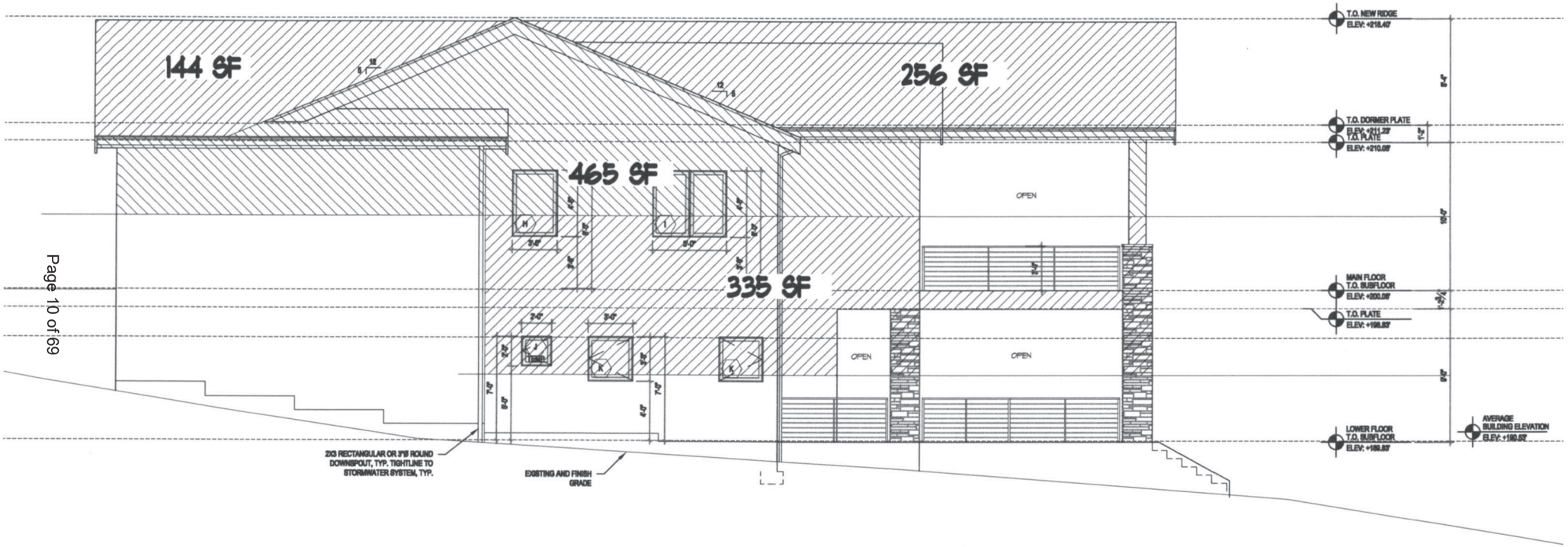

414 SF
570 SF
805 SF
GAS TERMINATION CAPS PER FIREPLACE MFRS.
NEW WOOD FRAMED CHIMNEY W/ SIDING TO MATCH ELEVATIONS OVER NEW GAS FIREPLACE FLUES
CRICKET
T.O. NEW RIDGE
ELEV: +218.40'
T.O. DORMER PLATE
ELEV: +211.23'
T.O. PLATE
ELEV: +210.06'
MAIN FLOOR
T.O. SUBFLOOR
ELEV: +200.06'
T.O. PLATE
ELEV: +198.83'
LOWER FLOOR
T.O. SUBFLOOR
ELEV: +189.83'
AVERAGE BUILDING ELEVA
ELEV: +190.55'
MAX DOWNHILL FACADE HEIGHT
29'-10"
3"Ø ROUND DOWNSPOUT, TIGHTLINE TO STORMWATER SYSTEM, TYP.
EGRESS
TEMP


144 SF
256 SF
465 SF
335 SF
OPEN
OPEN
OPEN
T.O. NEW RIDGE
ELEV: +218.40'
T.O. DORMER PLATE
ELEV: +211.23'
T.O. PLATE
ELEV: +210.06'
MAIN FLOOR
T.O. SUBFLOOR
ELEV: +200.06'
T.O. PLATE
ELEV: +198.83'
LOWER FLOOR
T.O. SUBFLOOR
ELEV: +189.83'
AVERAGE
BUILDING ELEVATION
ELEV: +190.92'
2X3 RECTANGULAR OR 3"Ø ROUND
DOWNSPOUT, TYP. TIGHTLINE TO
STORMWATER SYSTEM, TYP.
EXISTING AND FINISH
GRADE

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 17.97$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 16.56$

$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 19.16$

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 $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 12.22 \ \frac{lb}{ft^2}$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 11.26 \ \frac{lb}{ft^2}$

Front to Back Pressures:

Windward Roof $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 0.33 \ \frac{lb}{ft^2}$

Leeward Roof $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -9.77 \ \frac{lb}{ft^2}$

Leeward Wall $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -8.14 \ \frac{lb}{ft^2}$

Side to Side Pressures:

Windward Roof $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 4.40 \ \frac{lb}{ft^2}$

Leeward Roof $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -9.77 \ \frac{lb}{ft^2}$

Leeward Wall $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -6.84 \ \frac{lb}{ft^2}$

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

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

$p_{wr1} - p_{lr1} = 10.10 \ \frac{lb}{ft^2}$ $p_{ww1} - p_{lw1} = 20.37 \ \frac{lb}{ft^2}$ $p_{ww2} - p_{lw1} = 19.40 \ \frac{lb}{ft^2}$

$p_{wr2} - p_{lr2} = 14.17 \ \frac{lb}{ft^2}$ $p_{ww1} - p_{lw2} = 19.06 \ \frac{lb}{ft^2}$ $p_{ww2} - p_{lw2} = 18.10 \ \frac{lb}{ft^2}$

Wind Pressure at Main Roof (Front to Back):

$V_{1W} := (p_{wr1} - p_{lr1}) \cdot 414 \cdot ft^2 + (p_{ww1} - p_{lw1}) \cdot 570 \cdot ft^2 = 15787.98 \ lb$

Wind Pressure at Lower Floor (Front to Back):

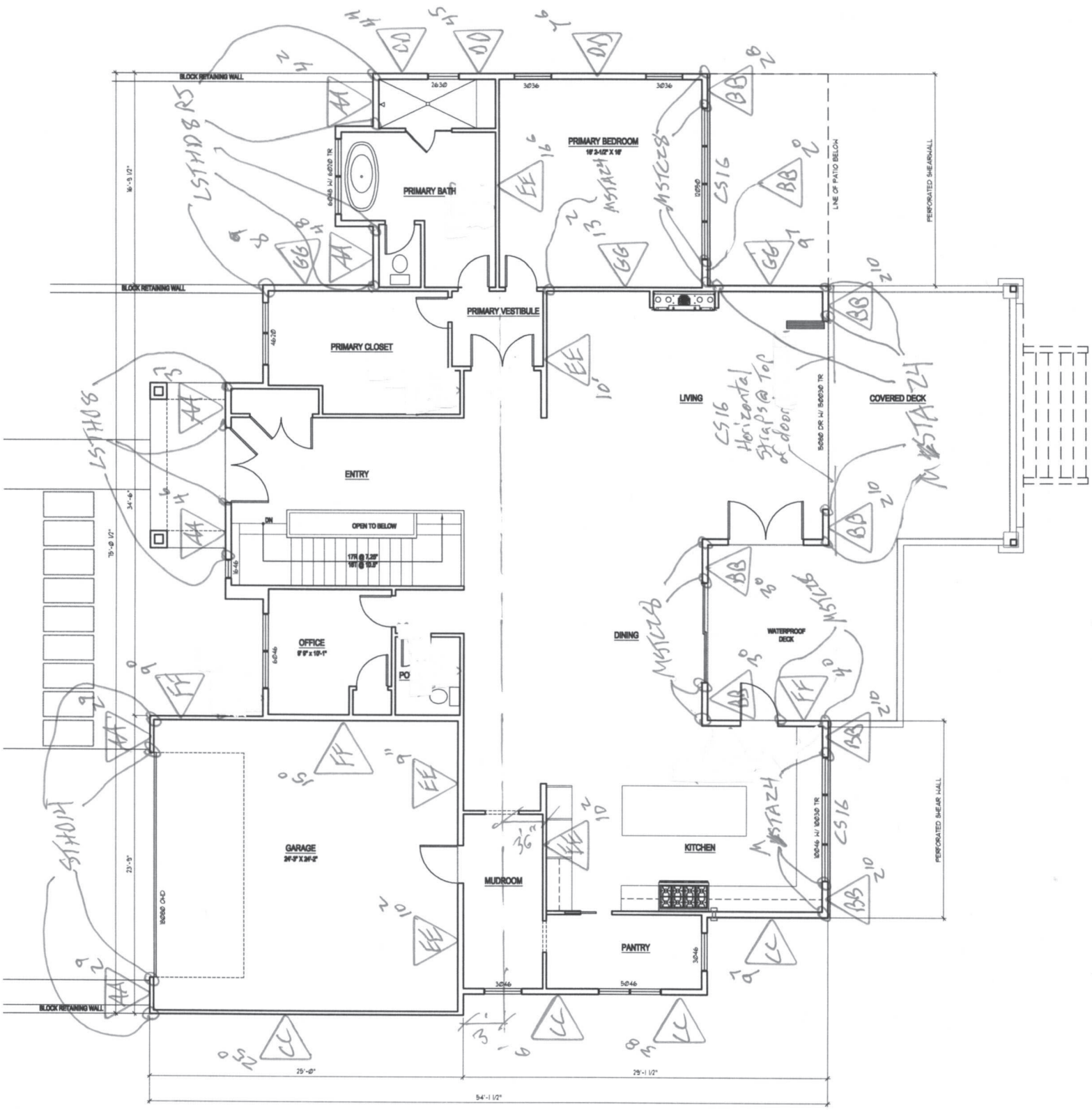
$V_{2W} := (p_{wr1} - p_{lr1}) \cdot 0 \cdot ft^2 + (p_{ww2} - p_{lw1}) \cdot 805 \cdot ft^2 = 15617.49 \ lb$

Wind Pressure at Main Roof (Side to Side):

$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 400 \cdot ft^2 + (p_{ww1} - p_{lw2}) \cdot 465 \cdot ft^2 = 14530.96 \ lb$

Wind Pressure at Lower Floor (Side to Side):

$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0 \cdot ft^2 + (p_{ww2} - p_{lw2}) \cdot 335 \cdot ft^2 = 6062.78 \ lb$


BLOCK RETAINING WALL
PRIMARY BEDROOM
PRIMARY BATH
PRIMARY VESTIBULE
PRIMARY CLOSET
LIVING
COVERED DECK
LINE OF PATIO BELOW
PERFORATED SHEARWALL
ENTRY
OPEN TO BELOW
OFFICE
PO
DINING
WATERPROOF DECK
KITCHEN
PANTRY
MUDROOM
GARAGE
PERFORATED SHEAR WALL
25'-0"
29'-1 1/2"
54'-1 1/2"

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WALL AA:

Story Shear due to Wind: $V_{3W} = 14530.96\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 28 \cdot ft$

Shear Wall Length:

$$Laa := \left(2 \cdot 2.75 \left(1.25 - 0.125 \left(\frac{10}{2.75}\right)\right) + 4.5 \left(1.25 - 0.125 \left(\frac{11}{4.5}\right)\right) + 3.58 \left(1.25 - 0.125 \left(\frac{11}{3.58}\right)\right) + 4.67 \left(1.25 - 0.125 \left(\frac{10}{4.67}\right)\right) + 4.17 \left(1.25 - 0.125 \left(\frac{10}{4.17}\right)\right)\right) \cdot ft$$

$Laa = 20.28\ ft$

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

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

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

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

$vaa = 111.49\ \frac{lb}{ft}$ $\frac{vaa}{C_o} = 111.49\ \frac{lb}{ft}$

$E_{aa} = 182.37\ \frac{lb}{ft}$ $\frac{E_{aa}}{C_o} = 182.37\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{aa} := 2.75 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRaa := \frac{W_{aa} \cdot L_{aa}}{2} = 178.75\ lb$

Chord Force:

$CFaa_w := \frac{vaa \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 1114.86\ lb$

$CFaa_s := \frac{E_{aa} \cdot L_{aa} \cdot Pt}{C_o \cdot L_{aa}} = 1823.70\ lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6 \cdot DLRaa = 1007.61\ lb$

$HDFaa_s := CFaa_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRaa = 1739.64\ lb$

Simpson LSTHD8/RJ at 8" stem walls & STHD14 at 6" stem walls

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vaa} = 1.46\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{aa}} = 0.89\ ft$

16d Sinkers @ 8" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vaa} = 12.34\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{aa}} = 7.55\ ft$

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

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Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 26 \cdot ft$

Shear Wall Length: $Lbb := \left(2 \cdot 2.83 + 2 \cdot 3 \left(1.25 - 0.125 \left(\frac{10}{3}\right)\right) + 2 \cdot 2.83 \left(\frac{10}{2.83}\right) + 2 \left(1.25 - 0.125 \left(\frac{5}{2}\right)\right) + 2.67\right) \cdot ft = 35.21\ ft$

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

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

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

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

$vbb = 59.62\ \frac{lb}{ft}$ $\frac{vbb}{C_o} = 59.62\ \frac{lb}{ft}$

$E_{bb} = 97.53\ \frac{lb}{ft}$ $\frac{E_{bb}}{C_o} = 97.53\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{bb} := 2.83 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRbb := \frac{W_{bb} \cdot L_{bb}}{2} = 183.95\ lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} = 596.20\ lb$

$CFbb_s := \frac{E_{bb} \cdot L_{bb} \cdot Pt}{C_o \cdot L_{bb}} = 975.27\ lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 485.83\ lb$

$HDFbb_s := CFbb_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRbb = 888.77\ lb$

Simpson LSTA24 to rim beams & MSTC28 a wall below

Horizontal Strap force at opening: $260\ plf \cdot 6\ ft = 1560.00\ lb$ Use Simpson CS16 coiled strap

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vbb} = 2.74\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 1.67\ ft$

16d Sinkers @ 16" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vbb} = 23.08\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{bb}} = 14.11\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL CC:

Story Shear due to Wind: $V_{1W} = 15787.98\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 75 \cdot ft$

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

Shear Wall Length: $Lcc := \left(25 + 6 + 3.67 \left(1.25 - 0.125 \left(\frac{10}{3.67}\right)\right)\right) \cdot ft = 34.34\ ft$

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

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

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

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

$vcc = 44.14\ \frac{lb}{ft}$ $\frac{vcc}{C_o} = 44.14\ \frac{lb}{ft}$

$E_{cc} = 66.46\ \frac{lb}{ft}$ $\frac{E_{cc}}{C_o} = 66.46\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{cc} := 3.67 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRcc := \frac{W_{cc} \cdot L_{cc}}{2} = 238.55\ lb$

Chord Force:

$CFcc_w := \frac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} = 441.40\ lb$

$CFcc_s := \frac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}} = 664.56\ lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6 \cdot DLRcc = 298.27\ lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRcc = 552.37\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vcc} = 3.70\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 2.46\ ft$

16d Sinkers @ 16" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vcc} = 31.17\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{cc}} = 20.71\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL DD:

Story Shear due to Wind: $V_{1W} = 15787.98\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 75 \cdot ft$

Distance between shear walls: $L_1 := 17 \cdot ft$

Shear Wall Length: $Ldd := \left(4.33\left(1.25 - 0.125\left(\frac{10}{4.33}\right)\right) + 4.42\left(1.25 - 0.125\left(\frac{10}{4.42}\right)\right) + 7.5\right) \cdot ft = 15.94\ ft$

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

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

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

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

$vdd = 67.36\ \frac{lb}{ft}$ $\frac{vdd}{C_o} = 67.36\ \frac{lb}{ft}$

$E_{dd} = 101.42\ \frac{lb}{ft}$ $\frac{E_{dd}}{C_o} = 101.42\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{dd} := 4.33 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRdd := \frac{W_{dd} \cdot L_{dd}}{2} = 281.45\ lb$

Chord Force:

$CFdd_w := \frac{vdd \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} = 673.62\ lb$

$CFdd_s := \frac{E_{dd} \cdot L_{dd} \cdot Pt}{C_o \cdot L_{dd}} = 1014.19\ lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6 \cdot DLRdd = 504.75\ lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRdd = 881.83\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vdd} = 2.42\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{dd}} = 1.61\ ft$

16d Sinkers @ 16" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vdd} = 20.43\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{dd}} = 13.57\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL EE:

Story Shear due to Wind: $V_{3W} = 14530.96\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$

Distance between shear walls: $L_1 := 28 \cdot ft$ $L_2 := 26 \cdot ft$

Shear Wall Length: $Lee := (10.17 + 9.92 + 10.17 + 10.08 + 16.5) \cdot ft = 56.84\ ft$

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

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

Wind Force: $vee := \frac{\frac{0.6 \cdot V_{3W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lee}$

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

$vee = 76.69\ \frac{lb}{ft}$ $\frac{vee}{C_o} = 76.69\ \frac{lb}{ft}$

$E_{ee} = 125.46\ \frac{lb}{ft}$ $\frac{E_{ee}}{C_o} = 125.46\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{ee} := 9.92 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

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

$DLRee := \frac{W_{ee} \cdot L_{ee}}{2} = 793.60\ lb$

Chord Force:

$CFee_w := \frac{vee \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}} = 766.94\ lb$

$CFee_s := \frac{E_{ee} \cdot L_{ee} \cdot Pt}{C_o \cdot L_{ee}} = 1254.57\ lb$

Holdown Force:

$HDFee_w := CFee_w - 0.6 \cdot DLRee = 290.78\ lb$

$HDFee_s := CFee_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRee = 881.37\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vee} = 2.13\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ee}} = 1.30\ ft$

16d Sinkers @ 16" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vee} = 17.94\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ee}} = 10.97\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL FF:

Story Shear due to Wind: $V_{1W} = 15787.98\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 75 \cdot ft$

Distance between shear walls: $L_1 := 24 \cdot ft$ $L_2 := 35 \cdot ft$

Shear Wall Length: $Lff := \left(9 + 15 + 4\left(1.25 - 0.125\left(\frac{11}{4}\right)\right)\right) \cdot ft = 27.63\ ft$

Percent full height sheathing: $\left(\frac{24 \cdot ft}{24.5 \cdot ft}\right) \cdot 100 = 97.96$

Max Opening Height = 10ft-0in, Therefore
$C_o := 0.96$ per AF&PA SDPWS Table 4.3.3.5

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

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

$vff = 134.88\ \frac{lb}{ft}$ $\frac{vff}{C_o} = 140.50\ \frac{lb}{ft}$

$E_{ff} = 203.07\ \frac{lb}{ft}$ $\frac{E_{ff}}{C_o} = 211.53\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{ff} := 4 \cdot ft$ Plate Height: $Pt := 11 \cdot ft$

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

$DLRff := \frac{W_{ff} \cdot L_{ff}}{2} = 340.00\ lb$

Chord Force:

$CFff_w := \frac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 1545.46\ lb$

$CFff_s := \frac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 2326.80\ lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6 \cdot DLRff = 1341.46\ lb$

$HDFff_s := CFff_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRff = 2166.91\ lb$

MSTC28 to rim beam

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PROJECT : **2434 73rd AVE SE**

Dead Load Resisting Overturning: $L_{ff} := 24 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_{ff} := (15 \cdot psf) \cdot 12 \cdot ft + (10 \cdot psf) \cdot Pt + (10 \cdot psf) \cdot 0 \cdot ft$ $DLRff := \frac{W_{ff} \cdot L_{ff}}{2} = 3360.00\ lb$

Chord Force:

$CFff_w := \frac{vff \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 1404.96\ lb$ $CFff_s := \frac{E_{ff} \cdot L_{ff} \cdot Pt}{C_o \cdot L_{ff}} = 2115.28\ lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6 \cdot DLRff = -611.04\ lb$ $HDFff_s := CFff_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRff = 535.18\ lb$

No Holdown Required

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vff} = 1.16\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{ff}} = 0.77\ ft$

16d Sinkers @ 16" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vff} = 9.79\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{ff}} = 6.51\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL GG:

Story Shear due to Wind: $V_{1W} = 15787.98\ lb$

Story Shear due to Seismic: $F_1 = 15672.51\ lb$

Bldg Width in direction of Load: $L_t := 75 \cdot ft$

Distance between shear walls: $L_1 := 35 \cdot ft$ $L_2 := 17 \cdot ft$

Shear Wall Length: $Lgg := (8.75 + 13.17 + 9.58) \cdot ft = 31.50\ ft$

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

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

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

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

$vgg = 104.25\ \frac{lb}{ft}$ $\frac{vgg}{C_o} = 104.25\ \frac{lb}{ft}$

$E_{gg} = 156.96\ \frac{lb}{ft}$ $\frac{E_{gg}}{C_o} = 156.96\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_{gg} := 9.58 \cdot ft$ Plate Height: $Pt := 11 \cdot ft$

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

$DLRgg := \frac{W_{gg} \cdot L_{gg}}{2} = 670.60\ lb$

Chord Force:

$CFgg_w := \frac{vgg \cdot L_{gg} \cdot Pt}{C_o \cdot L_{gg}} = 1146.76\ lb$

$CFgg_s := \frac{E_{gg} \cdot L_{gg} \cdot Pt}{C_o \cdot L_{gg}} = 1726.53\ lb$

Holdown Force:

$HDFgg_w := CFgg_w - 0.6 \cdot DLRgg = 744.40\ lb$

$HDFgg_s := CFgg_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRgg = 1411.17\ lb$

Simpson LSTHD8/RJ to foundation & MSTA24 to floor/rim beam

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vgg} = 1.57\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{gg}} = 1.04\ ft$

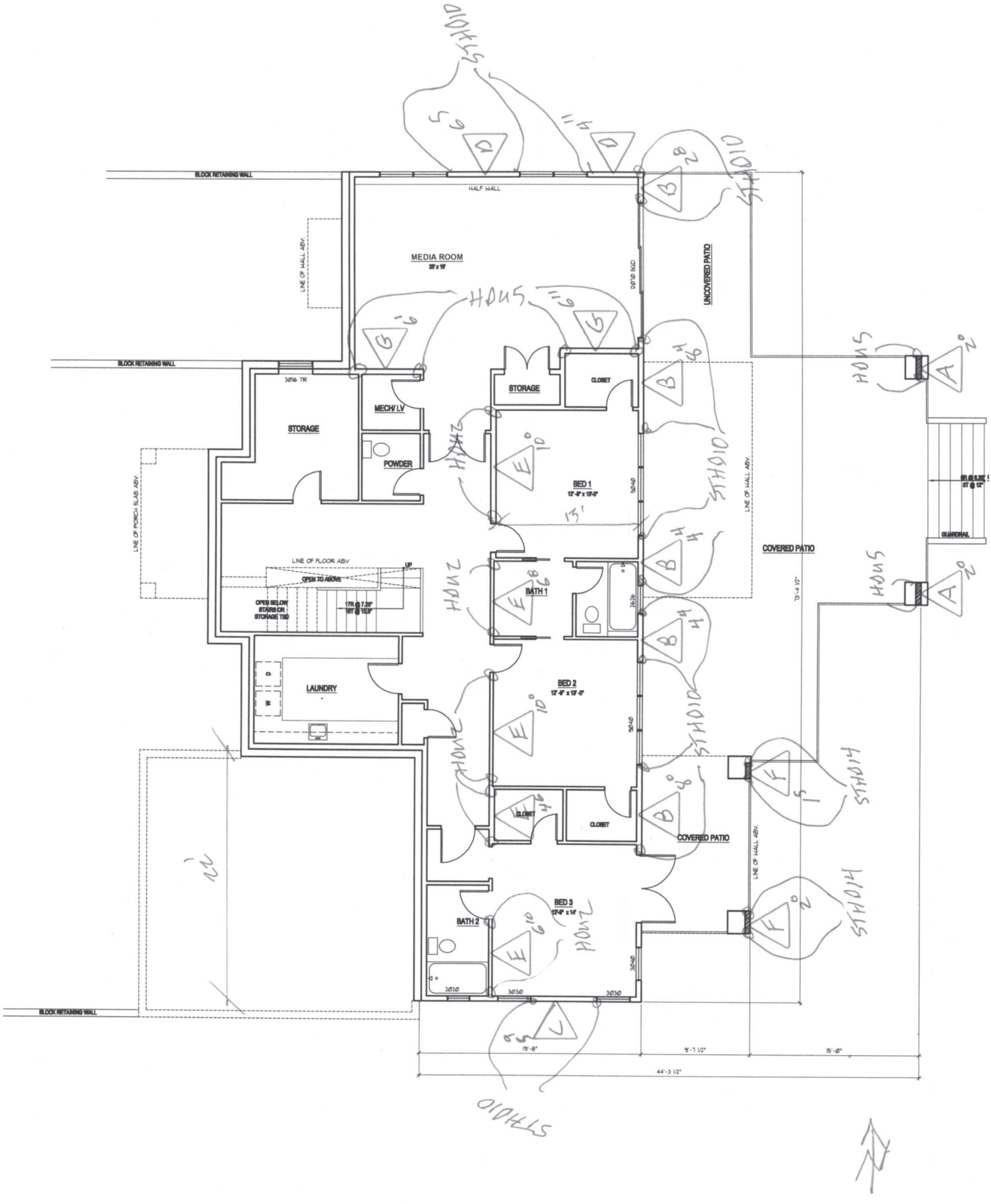
16d Sinkers @ 12" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vgg} = 13.20\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_{gg}} = 8.77\ ft$

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


BLOCK RETAINING WALL
HALF WALL
MEDIA ROOM
20' x 19'
LINE OF HALL ABV.
UNCOVERED PATIO
BLOCK RETAINING WALL
3016 TR
STORAGE
MECH/ LV
POWDER
STORAGE
CLOSET
BED 1
12'-8" x 13'-0"
LINE OF PORCH SLAB ABV.
LINE OF FLOOR ABV
OPEN TO ABOVE
UP
OPEN BELOW
STAIRS OR
STORAGE TBD
17R @ 7.25"
9T @ 10.5"
BATH 1
COVERED PATIO
LAUNDRY
BED 2
12'-6" x 13'-6"
CLOSET
CLOSET
COVERED PATIO
LINE OF HALL ABV.
BED 3
12'-6" x 14'
BATH 2
GUARDRAIL
13'-4 1/2"
19'-8"
9'-7 1/2"
15'-8"
44'-3 1/2"
BLOCK RETAINING WALL

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PROJECT : **2434 73rd AVE SE**

WALL A:

Story Shear due to Wind: $V_{4W} = 6062.78\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 44 \cdot ft$

Distance between shear walls: $L_1 := 25 \cdot ft$

Shear Wall Length: $La := (2 \cdot 2) \cdot ft = 4.00\ ft$

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

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

Wind Force: $va := \frac{\frac{10}{25}\left(vbb \cdot 5.67\ ft\right) + \left(\frac{0.6 \cdot V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$va = 292.16\ \frac{lb}{ft}$ $\qquad \frac{va}{C_o} = 292.16\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $\qquad E_a := \frac{\frac{10}{25}\left(E_{bb} \cdot 5.67\ ft\right) + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$E_a = 489.41\ \frac{lb}{ft}$ $\qquad \frac{E_a}{C_o} = 489.41\ \frac{lb}{ft}$

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

Restraint Panel Height = 10ft-0in Maximum
Restraint Panel Width = 2ft-0 in Minimum

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

Shear per Panel: $V_{s1} := \left(2 \cdot ft \cdot E_a\right) = 978.81\ lb$ O.K.

$V_{s2} := \left(2 \cdot ft \cdot va\right) = 584.32\ lb$ O.K.

Use Simpson HDU5 holdowns w/ SB5/8x24 anchors in lieu of STHD14
foundation straps due to location of portal wall on concrete plinth

Mark Myers, PE 07/01/2025

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PROJECT : **2434 73rd AVE SE**

WALL B:

Story Shear due to Wind: $V_{4W} = 6062.78\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 29 \cdot ft$

Distance between shear walls: $L_1 := 13 \cdot ft$ $L_2 := 10\ ft$

Shear Wall Length: $Lb := \left(8 + 2 \cdot 4.33 \left(1.25 - 0.125 \left(\frac{9}{4.33}\right)\right) + 8.33 + 2.67 \left(1.25 - 0.125 \left(\frac{9}{2.67}\right)\right)\right) \cdot ft = 27.12\ ft$

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

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

Wind Force: $vb := \frac{vbb \cdot Lbb + \left(\frac{0.6 \cdot V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lb}$

$vb = 130.60\ \frac{lb}{ft}$ $\frac{vb}{C_o} = 130.60\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_b := \frac{E_{bb} \cdot Lbb + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lb}$

$E_b = 216.00\ \frac{lb}{ft}$ $\frac{E_b}{C_o} = 216.00\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_b := 2.83 \cdot ft$

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

$W_b := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10 \cdot psf) \cdot 6.5 \cdot ft$

$DLRb := \frac{W_b \cdot L_b}{2} = 219.33\ lb$

Chord Force:

$CFb_w := \frac{vb \cdot L_b \cdot Pt}{C_o \cdot L_b} = 1175.37\ lb$

$CFb_s := \frac{E_b \cdot L_b \cdot Pt}{C_o \cdot L_b} = 1943.96\ lb$

$CFb_w + CFbb_w = 1771.56\ lb$

$CFb_s + CFbb_s = 2919.23\ lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1043.77\ lb$

$HDFb_s := CFb_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRb = 1840.82\ lb$

$HDFb_w + HDFbb_w = 1529.60\ lb$

$HDFb_s + HDFbb_s = 2729.59\ lb$

Simpson STHD10

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vb} = 1.25\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.76\ ft$

16d Sinkers @ 8" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vb} = 10.54\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_b} = 6.37\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL C:

Story Shear due to Wind: $V_{2W} = 15617.49\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 74 \cdot ft$

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

Shear Wall Length: $Lc := (5.75) \cdot ft = 5.75\ ft$

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

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

Wind Force: $vc := \frac{vcc \cdot 19.33\ ft + \left(\frac{0.6 \cdot V_{2W}}{L_t} \cdot \frac{L_1}{2}\right)}{Lc}$

$vc = 390.63\ \frac{lb}{ft}$ $\frac{vc}{C_o} = 390.63\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $E_c := \frac{E_{cc} \cdot 19.33\ ft + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Lc}$

$E_c = 381.42\ \frac{lb}{ft}$ $\frac{E_c}{C_o} = 381.42\ \frac{lb}{ft}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 532 plf
Seismic Capacity = 380 plf (OK, within 1%)

Dead Load Resisting Overturning: $L_c := 5.75 \cdot ft$

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

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

$DLRc := \frac{W_c \cdot L_c}{2} = 287.50\ lb$

Chord Force:

$CFc_w := \frac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c} = 3515.68\ lb$

$CFc_s := \frac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c} = 3432.78\ lb$

$CFc_w + CFcc_w = 3957.08\ lb$

$CFc_s + CFcc_s = 4097.33\ lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 3343.18\ lb$

$HDFc_s := CFc_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRc = 3297.57\ lb$

$HDFc_w + HDFcc_w = 3641.45\ lb$

$HDFc_s + HDFcc_s = 3849.95\ lb$

Simpson STHD14

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vc} = 0.42\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 0.43\ ft$

16d Sinkers @ 4" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vc} = 3.52\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_c} = 3.61\ ft$

5/8" dia. anchors @ 42" o.c.

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PROJECT : **2434 73rd AVE SE**

WALL D:

Story Shear due to Wind: $V_{2W} = 15617.49\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 74 \cdot ft$

Distance between shear walls: $L_1 := 17 \cdot ft$

Shear Wall Length: $Ld := (6.42 + 4.92) \cdot ft = 11.34\ ft$

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

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

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

$vd = 189.59\ \frac{lb}{ft}$ $\frac{vd}{C_o} = 189.59\ \frac{lb}{ft}$

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

$E_d = 204.45\ \frac{lb}{ft}$ $\frac{E_d}{C_o} = 204.45\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_d := 4.92 \cdot ft$

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

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

$DLRd := \frac{W_d \cdot L_d}{2} = 246.00\ lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d} = 1706.29\ lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d} = 1840.03\ lb$

$CFd_w + CFdd_w = 2379.91\ lb$

$CFd_s + CFdd_s = 2854.22\ lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6 \cdot DLRd = 1558.69\ lb$

$HDFd_s := CFd_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRd = 1724.35\ lb$

$HDFd_w + HDFdd_w = 2063.44\ lb$

$HDFd_s + HDFdd_s = 2606.18\ lb$

Simpson STHD10

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vd} = 0.86\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 0.80\ ft$

16d Sinkers @ 8" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vd} = 7.26\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_d} = 6.73\ ft$

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

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PROJECT : **2434 73rd AVE SE**

WALL E:

Story Shear due to Wind: $V_{4W} = 6062.78\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 29 \cdot ft$

Distance between shear walls: $L_1 := 13 \cdot ft$ $L_2 := 7 \cdot ft$

Shear Wall Length: $Le := (6.83 + 4.5 + 10 + 6.67 + 10) \cdot ft = 38.00\ ft$

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

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

Wind Force: $ve := \frac{vee \cdot Lee + \left(\frac{0.6 \cdot V_{4W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Le}$

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

$ve = 147.73\ \frac{lb}{ft}$ $\frac{ve}{C_o} = 147.73\ \frac{lb}{ft}$

$E_e = 243.12\ \frac{lb}{ft}$ $\frac{E_e}{C_o} = 243.12\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_e := 4.5 \cdot ft$

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

$W_e := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10 \cdot psf) \cdot 9 \cdot ft$

$DLRe := \frac{W_e \cdot L_e}{2} = 405.00\ lb$

Chord Force:

$CFe_w := \frac{ve \cdot L_e \cdot Pt}{C_o \cdot L_e} = 1329.55\ lb$

$CFe_s := \frac{E_e \cdot L_e \cdot Pt}{C_o \cdot L_e} = 2188.10\ lb$

$CFe_w + CFee_w = 2096.49\ lb$

$CFe_s + CFee_s = 3442.68\ lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6 \cdot DLRe = 1086.55\ lb$

$HDFe_s := CFe_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRe = 1997.65\ lb$

$HDFe_w + HDFee_w = 1377.33\ lb$

$HDFe_s + HDFee_s = 2879.02\ lb$

Simpson HDU2 w/ epoxied 5/8 diameter all-thread anchor embedded 6" into FOOTING

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{ve} = 1.10\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_e} = 0.67\ ft$

16d Sinkers @ 8" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{ve} = 9.31\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_e} = 5.66\ ft$

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

Mark Myers, PE 07/01/2025

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PROJECT : **2434 73rd AVE SE**

WALL F:

Story Shear due to Wind: $V_{4W} = 6062.78\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 29 \cdot ft$

Distance between shear walls: $L_1 := 10 \cdot ft$

Shear Wall Length: $Lf := (2 + 1.42) \cdot ft = 3.42\ ft$

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

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

Wind Force: $vf := \frac{vbb \cdot 5.67\ ft + \left(\frac{0.6 \cdot V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{Lf}$

$vf = 282.23\ \frac{lb}{ft}$ $\qquad \frac{vf}{C_o} = 282.23\ \frac{lb}{ft}$

Seismic Force: $\rho := 1.3$ $\qquad E_f := \frac{E_{bb} \cdot 5.67\ ft + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Lf}$

$E_f = 469.83\ \frac{lb}{ft}$ $\qquad \frac{E_f}{C_o} = 469.83\ \frac{lb}{ft}$

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

Restraint Panel Height = 10ft-0in Maximum
Restraint Panel Width = 2ft-0 in Minimum

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

Shear per Panel: $V_{s1} := (2 \cdot ft \cdot E_f) = 939.66\ lb$ O.K.

$V_{s2} := (2 \cdot ft \cdot vf) = 564.46\ lb$ O.K.

Restraint Panel Height = 10ft-0in Maximum
Restraint Panel Width = 1ft-5 in Minimum

Allowable Shear per Panel = 688 lbs Seismic & 963 lbs Wind

Shear per Panel: $V_{s1} := (1.42 \cdot ft \cdot E_f) = 667.16\ lb$ O.K.

$V_{s2} := (1.42 \cdot ft \cdot vf) = 400.77\ lb$ O.K.

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PROJECT : **2434 73rd AVE SE**

WALL G:

Story Shear due to Wind: $V_{2W} = 15617.49\ lb$

Story Shear due to Seismic: $F_2 = 6716.75\ lb$

Bldg Width in direction of Load: $L_t := 74 \cdot ft$

Distance between shear walls: $L_1 := 17 \cdot ft$ $L_2 := 34 \cdot ft$

Shear Wall Length: $Lg := (6.08 + 6.92) \cdot ft = 13.00\ ft$

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

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

Wind Force: $vg := \frac{vgg \cdot 22.75\ ft + \left(\frac{0.6 \cdot V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg}$

Seismic Force: $\rho := 1.0$ $E_g := \frac{E_{gg} \cdot 22.75\ ft + \left(\rho \cdot \frac{0.7 \cdot F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lg}$

$vg = 430.83\ \frac{lb}{ft}$ $\frac{vg}{C_o} = 430.83\ \frac{lb}{ft}$

$E_g = 399.31\ \frac{lb}{ft}$ $\frac{E_g}{C_o} = 399.31\ \frac{lb}{ft}$

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

Dead Load Resisting Overturning: $L_g := 6.08 \cdot ft$

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

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

$DLRg := \frac{W_g \cdot L_g}{2} = 304.00\ lb$

Chord Force:

$CFg_w := \frac{vg \cdot L_g \cdot Pt}{C_o \cdot L_g} = 3877.43\ lb$

$CFg_s := \frac{E_g \cdot L_g \cdot Pt}{C_o \cdot L_g} = 3593.75\ lb$

$CFg_w + CFgg_w = 5024.19\ lb$

$CFg_s + CFgg_s = 5320.28\ lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6 \cdot DLRg = 3695.03\ lb$

$HDFg_s := CFg_s - (0.6 - 0.14 \cdot S_{DS}) \cdot DLRg = 3450.79\ lb$

$HDFg_w + HDFgg_w = 4439.43\ lb$

$HDFg_s + HDFgg_s = 4861.96\ lb$

Simpson HDU5 w/ epoxied 5/8 diameter all-thread anchor embedded 6" into FOOTING

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

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

$\frac{(Z_N \cdot C_D \cdot C_o)}{vg} = 0.38\ ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_g} = 0.41\ ft$

16d Sinkers @ 4" o.c.

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

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

$\frac{(Z_B \cdot C_D \cdot C_o)}{vg} = 3.19\ ft$ $\frac{(Z_B \cdot C_D \cdot C_o)}{E_g} = 3.45\ ft$

5/8" dia. anchors @ 36" o.c.

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PROJECT : **2434 73rd AVE SE**

Diaphragm Shear Check:

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

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

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

Wall Lines AA:

$vaa \cdot \frac{Laa}{75 \cdot ft} = 30.14 \ \frac{lb}{ft}$ $E_{aa} \cdot \frac{Laa}{75 \cdot ft} = 49.30 \ \frac{lb}{ft}$

Wall Lines BB:

$vbb \cdot \frac{Lbb}{70 \cdot ft} = 29.98 \ \frac{lb}{ft}$ $E_{bb} \cdot \frac{Lbb}{70 \cdot ft} = 49.05 \ \frac{lb}{ft}$

Wall Lines CC:

$vcc \cdot \frac{Lcc}{54 \cdot ft} = 28.07 \ \frac{lb}{ft}$ $E_{cc} \cdot \frac{Lcc}{54 \cdot ft} = 42.26 \ \frac{lb}{ft}$

Wall Lines DD:

$vdd \cdot \frac{Ldd}{27 \cdot ft} = 39.76 \ \frac{lb}{ft}$ $E_{dd} \cdot \frac{Ldd}{27 \cdot ft} = 59.87 \ \frac{lb}{ft}$

Wall Lines A:

$\frac{va \cdot La}{22 \cdot ft} = 53.12 \ \frac{lb}{ft}$ $\frac{E_a \cdot La}{22 \cdot ft} = 88.98 \ \frac{lb}{ft}$

Wall Lines EE:

$vee \cdot \frac{Lee}{73 \cdot ft} = 59.72 \ \frac{lb}{ft}$ $E_{ee} \cdot \frac{Lee}{73 \cdot ft} = 97.68 \ \frac{lb}{ft}$

Wall Lines FF:

$vff \cdot \frac{Lff}{54 \cdot ft} = 69.00 \ \frac{lb}{ft}$ $E_{ff} \cdot \frac{Lff}{54 \cdot ft} = 103.88 \ \frac{lb}{ft}$

Wall Lines EE:

$vgg \cdot \frac{Lgg}{45 \cdot ft} = 72.98 \ \frac{lb}{ft}$ $E_{gg} \cdot \frac{Lgg}{45 \cdot ft} = 109.87 \ \frac{lb}{ft}$

Wall Lines F:

$\frac{vf \cdot Lf}{16 \cdot ft} = 60.33 \ \frac{lb}{ft}$ $\frac{E_f \cdot Lf}{16 \cdot ft} = 100.43 \ \frac{lb}{ft}$

Load from Diaphragm to Rim: Load Rim to Top Plate:

Wall Lines B:

$\frac{vb \cdot Lb - vbb \cdot Lbb}{73 \cdot ft} = 19.76 \ \frac{lb}{ft}$ $\frac{E_b \cdot Lb - E_{bb} \cdot Lbb}{73 \cdot ft} = 33.20 \ \frac{lb}{ft}$ $\frac{vb \cdot Lb}{73 \cdot ft} = 48.51 \ \frac{lb}{ft}$ $\frac{E_b \cdot Lb}{73 \cdot ft} = 80.24 \ \frac{lb}{ft}$

Wall Lines C:

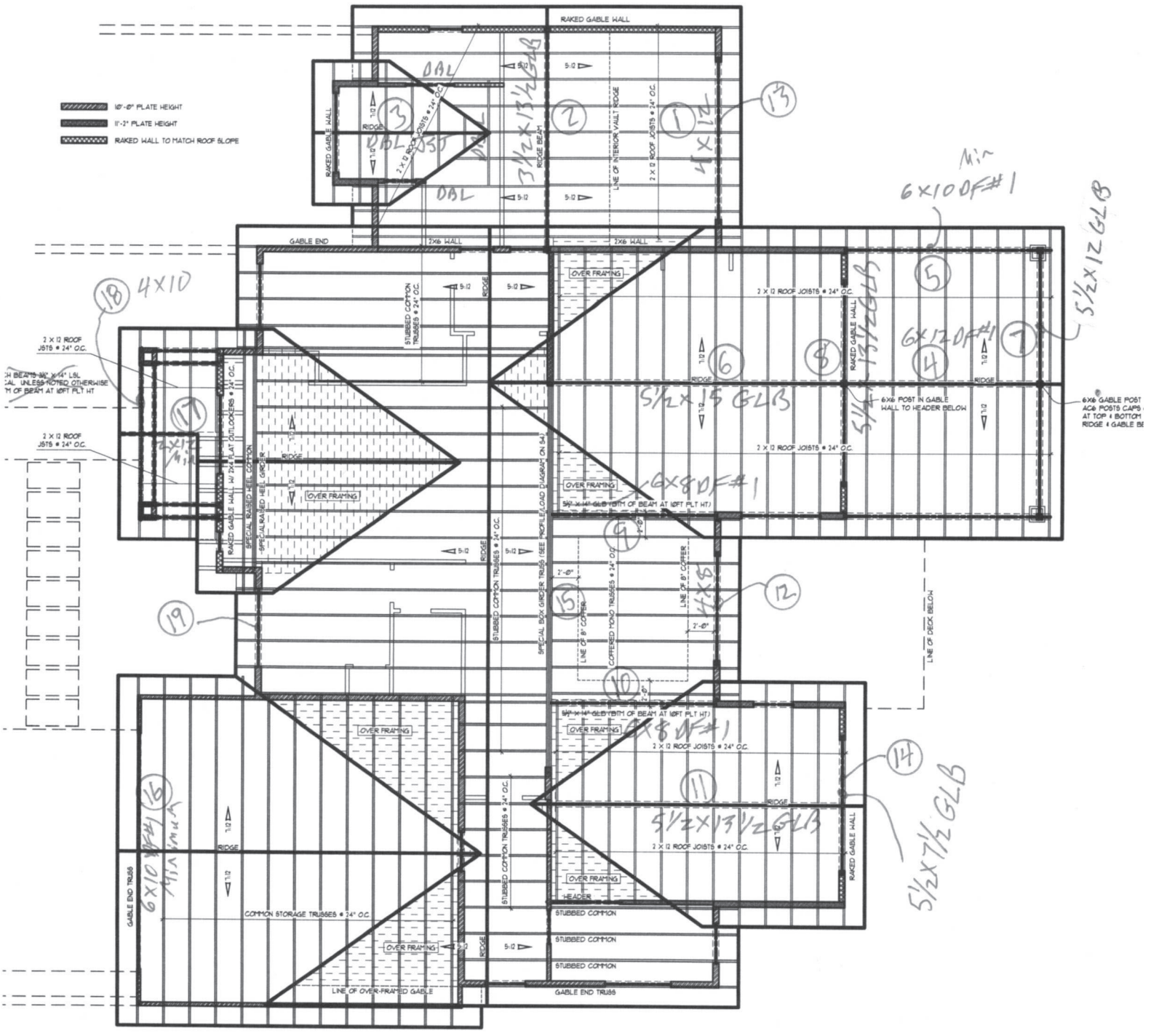
$\frac{vc \cdot Lc - vcc \cdot 19.33 \ ft}{20 \cdot ft} = 69.65 \ \frac{lb}{ft}$ $\frac{E_c \cdot Lc - E_{cc} \cdot 19.33 \ ft}{20 \cdot ft} = 45.43 \ \frac{lb}{ft}$ $\frac{vc \cdot Lc}{20 \cdot ft} = 112.31 \ \frac{lb}{ft}$ $\frac{E_c \cdot Lc}{20 \cdot ft} = 109.66 \ \frac{lb}{ft}$

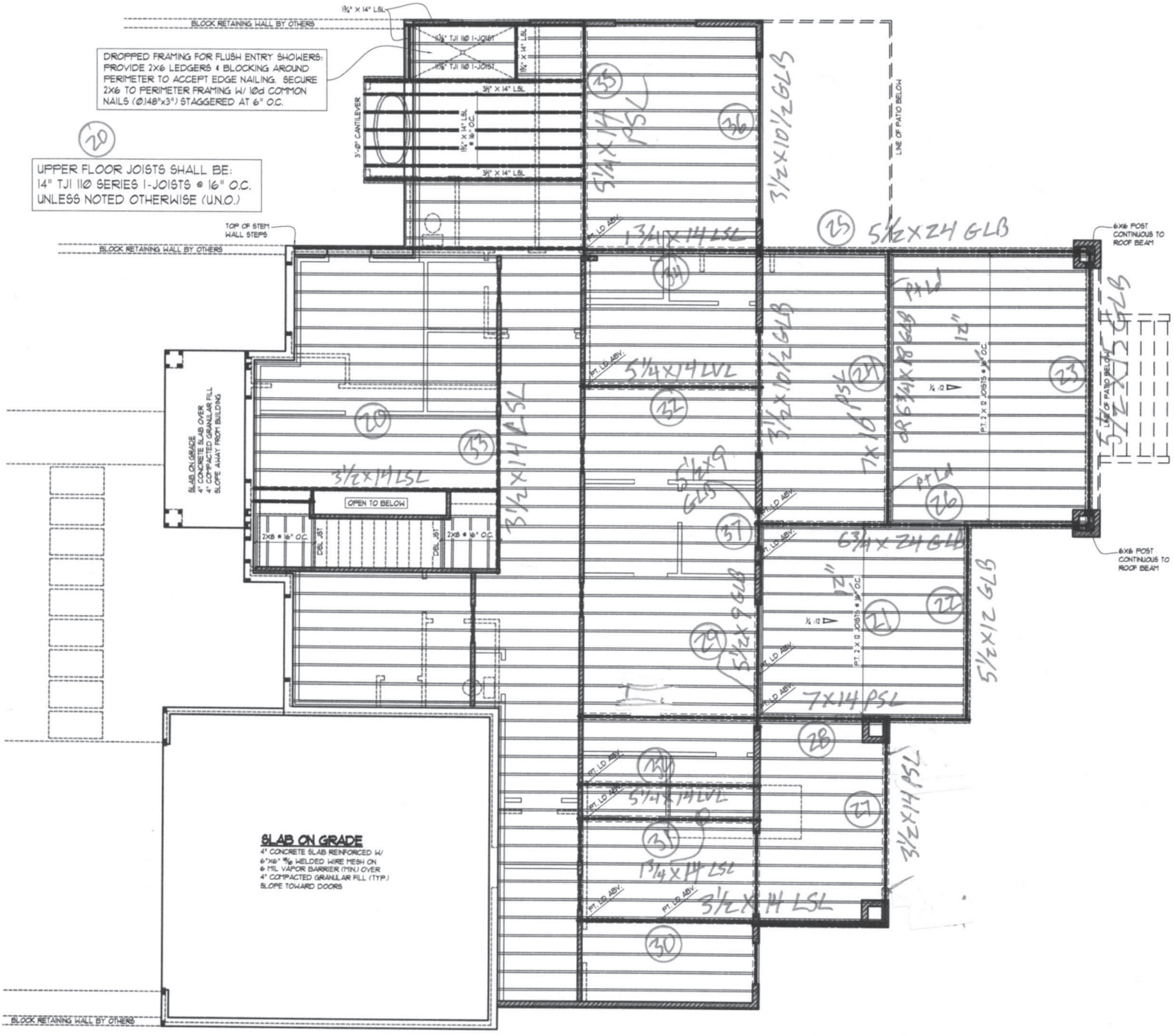
Wall Lines D:

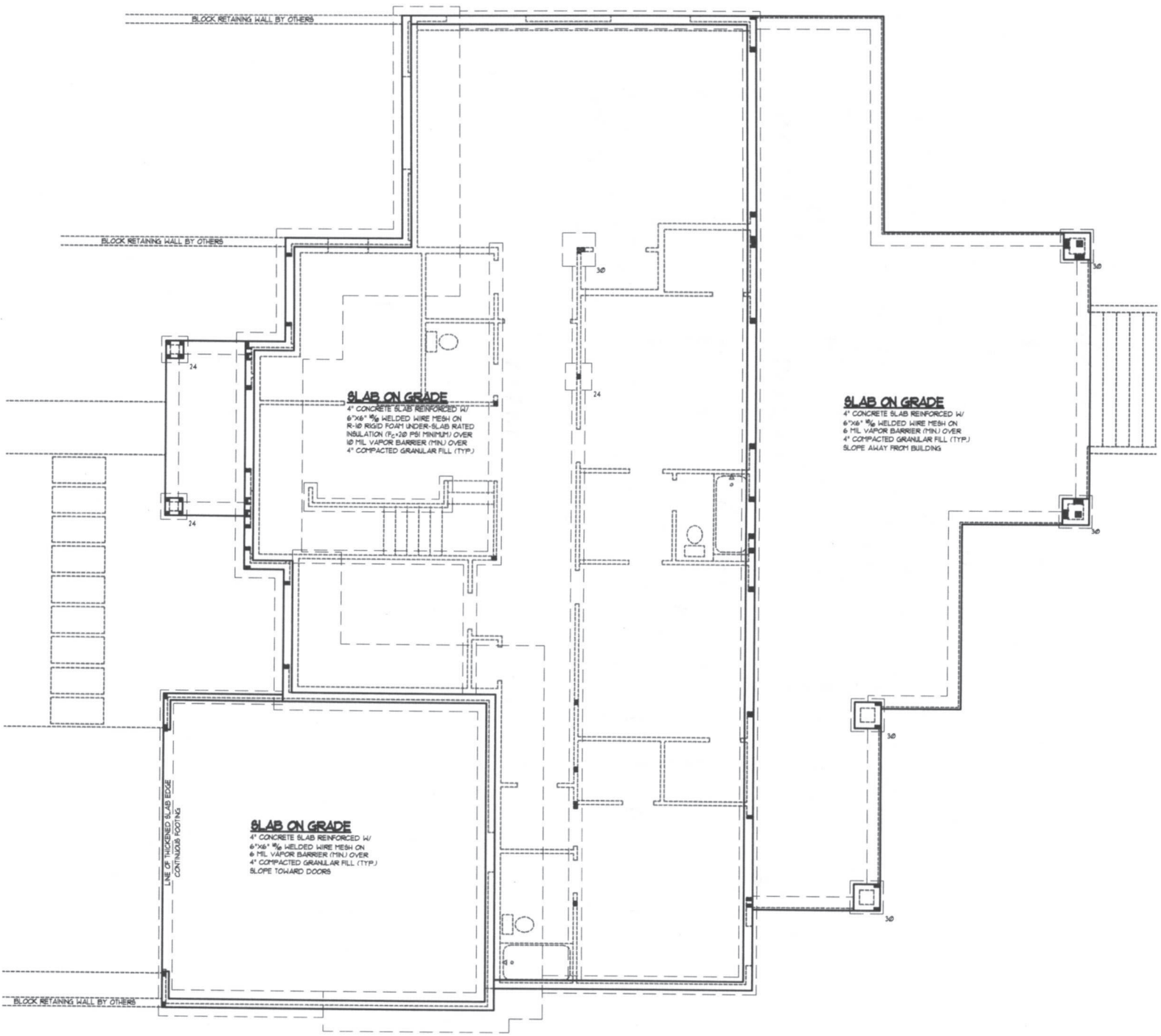
$\frac{vd \cdot Ld - vdd \cdot Ldd}{26 \cdot ft} = 41.40 \ \frac{lb}{ft}$ $\frac{E_d \cdot Ld - E_{dd} \cdot Ldd}{26 \cdot ft} = 27.00 \ \frac{lb}{ft}$ $\frac{vd \cdot Ld}{26 \cdot ft} = 82.69 \ \frac{lb}{ft}$ $\frac{E_d \cdot Ld}{26 \cdot ft} = 89.17 \ \frac{lb}{ft}$

Wall Line E:

$\frac{ve \cdot Le - vee \cdot Lee}{73 \cdot ft} = 17.18 \ \frac{lb}{ft}$ $\frac{E_e \cdot Le - E_{ee} \cdot Lee}{73 \cdot ft} = 28.87 \ \frac{lb}{ft}$ $\frac{ve \cdot Le}{52 \cdot ft} = 107.95 \ \frac{lb}{ft}$ $\frac{E_e \cdot Le}{52 \cdot ft} = 177.67 \ \frac{lb}{ft}$


10'-0" PLATE HEIGHT
11'-2" PLATE HEIGHT
RAKED WALL TO MATCH ROOF SLOPE
RAKED GABLE WALL
GABLE END
2X6 WALL
2 X 12 ROOF JOISTS @ 24" O.C.
LINE OF INTERIOR VAULT RIDGE
RIDGE BEAM
STUBBED COMMON TRUSSES @ 24" O.C.
OVER FRAMING
RAKED GABLE WALL W/ 2X6 FLAT OUTLOOKERS @ 24" O.C.
SPECIAL RAISED HEEL GIRDER
SPECIAL BOX GIRDER TRUSS (SEE PROFILE LOAD DIAGRAM ON S4)
COFFERED MONO TRUSSES @ 24" O.C.
LINE OF 8' COFFER
LINE OF DECK BELOW
5½" X 14" GLB (BTM OF BEAM AT 10FT PLT HT)
6X6 POST IN GABLE WALL TO HEADER BELOW
COMMON STORAGE TRUSSES @ 24" O.C.
GABLE END TRUSS
LINE OF OVER-FRAMED GABLE
HEADER
STUBBED COMMON
5½X15 GLB
5½X12 GLB
5½X13½ GLB
5½X7½ GLB
6X10 DF#1
6X12 DF#1
6X8 DF#1
4X10
4X12
4X8


DROPPED FRAMING FOR FLUSH ENTRY SHOWERS: PROVIDE 2X6 LEDGERS & BLOCKING AROUND PERIMETER TO ACCEPT EDGE NAILING. SECURE 2X6 TO PERIMETER FRAMING W/ 10d COMMON NAILS (Ø.148"x3") STAGGERED AT 6" O.C.
20
UPPER FLOOR JOISTS SHALL BE: 14" TJI 110 SERIES I-JOISTS @ 16" O.C. UNLESS NOTED OTHERWISE (U.N.O.)
BLOCK RETAINING WALL BY OTHERS
TOP OF STEM WALL STEPS
3'-0" CANTILEVER
LINE OF PATIO BELOW
OPEN TO BELOW
SLAB ON GRADE 4" CONCRETE SLAB OVER 4" COMPACTED GRANULAR FILL SLOPE AWAY FROM BUILDING
6X6 POST CONTINUOUS TO ROOF BEAM
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/ 6"X6" 9/9 WELDED WIRE MESH ON 6 MIL VAPOR BARRIER (MIN.) OVER 4" COMPACTED GRANULAR FILL (TYP.) SLOPE TOWARD DOORS


BLOCK RETAINING WALL BY OTHERS
BLOCK RETAINING WALL BY OTHERS
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" W1.4 WELDED WIRE MESH ON
R-10 RIGID FOAM UNDER-SLAB RATED
INSULATION (F'c=20 PSI MINIMUM) OVER
10 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" W1.4 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)
SLOPE AWAY FROM BUILDING
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" W1.4 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED GRANULAR FILL (TYP.)
SLOPE TOWARD DOORS
LINE OF THICKENED SLAB EDGE
CONTINUOUS FOOTING
BLOCK RETAINING WALL BY OTHERS
24
24
30
30
30
30
30
45°
45°

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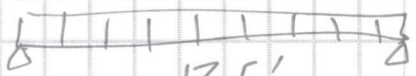
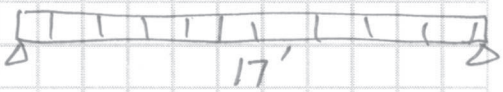
① $w_D = 15 psf$
$w_S = 25 psf$



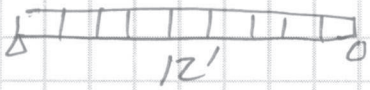
2X10 @ 24" o.c. Min

② $w_D = 15 psf \left(27'/2\right) = 202.5 plf$
$w_S = 25 psf \left(27'/2\right) = 337.5 plf$



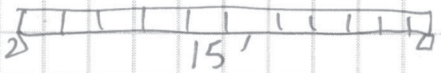
3½X13½ GLB

③ $w_D = 15 psf \left(8'/2\right) = 60 plf$
$w_S = 25 psf \left(8'/2\right) = 100 plf$



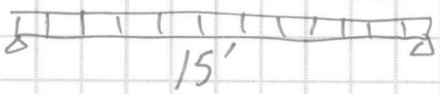
(2) 2X10 Min.

④ $w_D = 15 psf \left(20.5'/2\right) = 153.75 plf$
$w_S = 25 psf \left(20.5'/2\right) = 256.25 plf$



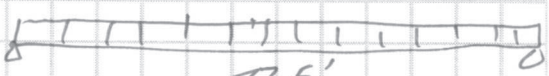
6X12 DF #1

⑤ $w_D = 15 psf \left(13'/2\right) = 97.5 plf$
$w_S = 25 psf \left(13'/2\right) = 162.5 plf$



6X10 DF #1
Minimum

⑥ $w_D = 15 psf \left(20.5'/2\right) = 153.75 plf$
$w_S = 25 psf \left(20.5'/2\right) = 256.25 plf$



5½"X15" GLB

FOR RKK
JOB 2434 73rd

DATE 6-10-25
BY MM

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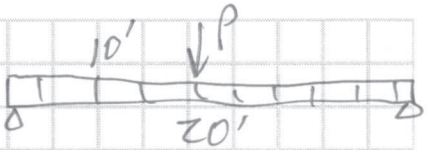
(7) w_D = self

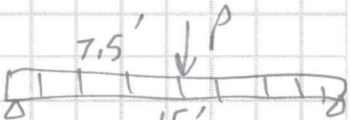
$P = 1200^{\#}DL + 2000^{\#}SL$ from (4)



5½x12 GLB

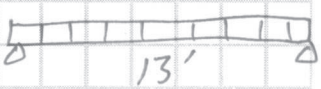
(8) $w_D = 12psf(5') = 60plf$

$P_1 = 1154^{\#}DL + 1922^{\#}SL$ from (4)
$P_2 = 1730^{\#}DL + 2890^{\#}SL$ from (6)



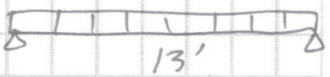
5½x13½ GLB

(9) $w_D = 15psf(12'/2) = 90plf$
$w_S = 25psf(12'/2) = 150plf$



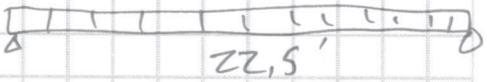
6x8 DF#1
Minimum

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



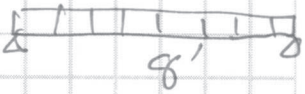
6x8 DF#1
Minimum

(11) $w_D = 15psf(16'/2) = 120plf$
$w_S = 25psf(16'/2) = 200plf$



5½x13½ GLB

(12) $w_D = 15psf(16'/2) = 120plf$
$w_S = 25psf(16'/2) = 200plf$



4x8

FOR RKK
JOB 2434 73rd

DATE 6-10-25
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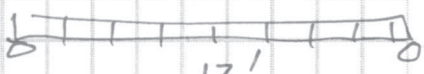
myengineer@centurytel.net

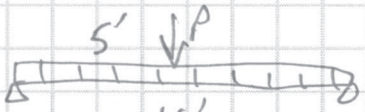
(13) $w_D = 15psf(16'/2) = 120plf$
$w_S = 25psf(16'/2) = 200plf$



4x12

(14) $w_D = 12psf(5') = 60plf$

$P = 1350\#DL + 2250\#SL$ from (11)



5½x7½ GLB

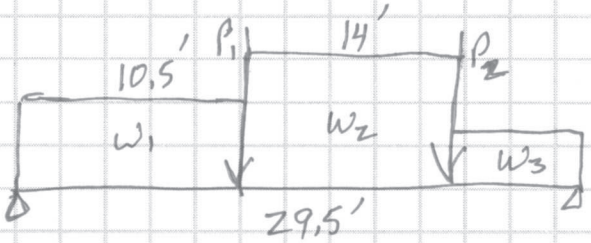
(15) $w_{D1} = 15psf(22.5'/2) = 168.75plf$
$w_{S1} = 25psf(22.5'/2) = 281.25plf$

$w_{D2} = 15psf(35'/2) = 262.5plf$
$w_{S2} = 25psf(35'/2) = 437.5plf$

$w_{D3} = 15psf(7'/2) = 52.5plf$
$w_{S3} = 25psf(7'/2) = 87.5plf$

$P_1 = 565\#DL + 975\#SL$ from (9)

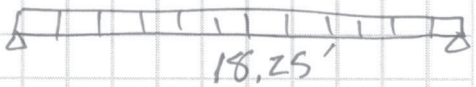
$P_2 = 440\#DL + 735\#SL$ from (10)



Load check for Reactions & Beam option.

5½x25½ GLB

(16) $w_D = 15psf(3') + 12psf(5') = 105plf$
$w_S = 25psf(3') = 75plf$



6x10 DF #1 Minimum

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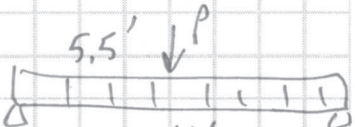
(17) $w_D = 15psf\left(\frac{13'}{2}\right) = 97.5plf$
$w_S = 25psf\left(\frac{13'}{2}\right) = 162.5plf$

2x8 DF#2
Minimum

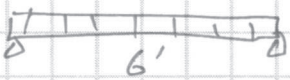
(18) $w_D = 12psf(4') = 48plf$ + self

$P = 440\#DL + 730\#SL$ from (17)



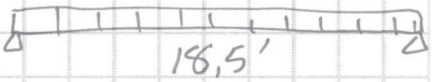
4x10 DF#2

(19) $w_D = 15psf\left(\frac{25'}{2}\right) = 187.5plf$
$w_S = 25psf\left(\frac{25'}{2}\right) = 312.5plf$



4x8

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

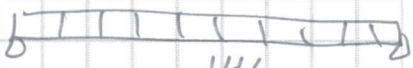


14" TJI 110 @ 16" o.c.

(21) $w_D = 25psf$
$w_L = 60psf$
$w_S = 25psf$

2x12 DF#2
@ 12" o.c.

(22) $w_D = 25psf\left(\frac{16'}{2}\right) = 200plf$
$w_L = 60psf\left(\frac{16'}{2}\right) = 480plf$
$w_S = 25psf\left(\frac{16'}{2}\right) = 200plf$



5 1/2 x 12 GLB

FOR RKK
JOB 2434 T3rd

DATE 6-10-25
BY MM

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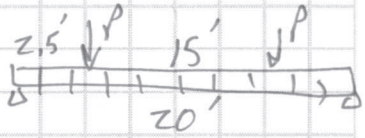
(23) $w_D = 25psf(15'/2) = 187.5plf$
$w_L = 60psf(15'/2) = 450plf$

5½x15 GLB

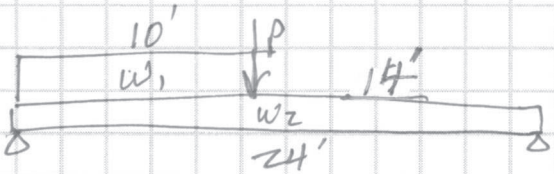
(24) $w_D = 15psf(10'/2) + 12psf(11') + 25psf(15'/2) = 395plf$
$w_L = 40psf(10'/2) + 60psf(15'/2) = 650plf$



$P = 1560\#DL + 2410\#SL$ from (8)

6¾x18 GLB
OR
7x16 PSL

(25) $w_{D1} = 15psf(13'/2) + 12psf(11') = 230plf$
$w_{S1} = 25psf(13'/2) = 162.5plf$

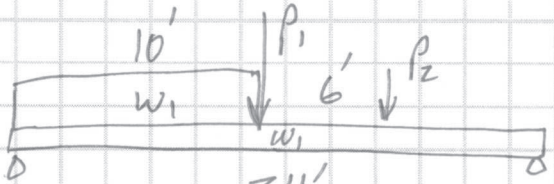


$w_{D2} = self$

$P = 5510\#DL + 6500\#LL + 2410\#SL$ from (24)

5½x24 GLB

(26) $w_{D1} = 230plf$
$w_{S1} = 162.5plf$



$w_{D2} = self$

$P_1 = 5510\#DL + 6500\#LL + 2410\#SL$ from (24)

$P_2 = 1400\#DL + 3360\#LL + 1400\#SL$ from (22)

6¾x24 GLB

FOR RKK
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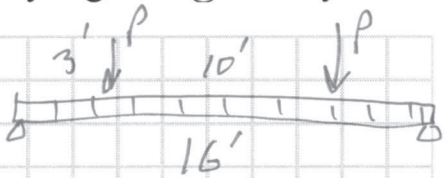
(27) $w_D = 15psf(10'/2 + 3') + 12psf(13') = 276plf$
$w_L = 40psf(10'/2) = 200plf$
$w_S = 25psf(3') = 75plf$

$P_1 = 980\#DL + 1130\#SL$ from (14)



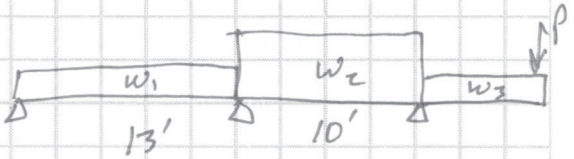
3½x14 PSL

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

$w_{D2} = 15psf(8'/2 + 1') + 12psf(11') = 207plf$
$w_{L2} = 40plf$
$w_{S2} = 25psf(8'/2) = 100plf$

$w_{D3} = 25plf$
$w_{L3} = 60plf$
$w_{S3} = 25plf$

$P = 1400\#DL + 3360\#LL + 1400\#SL$ from (22)



7x14 PSL

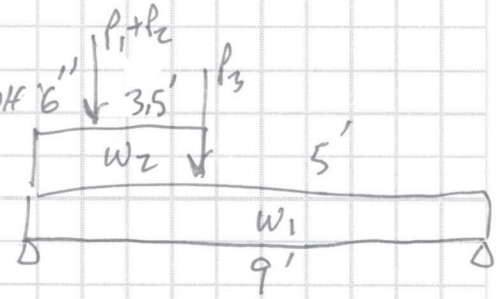
(29) $w_{D1} = 15psf(13'/2) + 25psf(15'/2) + 12psf(11') = 417plf$
$w_{L1} = 40psf(13'/2) + 60psf(15'/2) = 710plf$
$w_{S1} = 25psf(15'/2) = 187.5plf$

$w_{D2} = 15psf(16'/2) = 120plf$
$w_{S2} = 25psf(16'/2) = 200plf$

$P_1 = 171\#DL + (-2280\#)LL + (-600\#)SL$ from (28)

$P_2 = 440\#DL + 730\#SL$ from (10)

$P_3 = 480\#DL + 800\#SL$ from (12)



5½x9 GLB

FOR RKK
JOB 2434 73rd

DATE 6-10-25
BY

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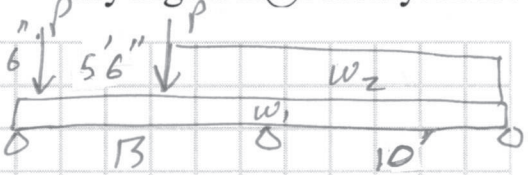
(30) $w_{D_1} = 15 plf$
$w_{L_1} = 40 plf$

$w_{D_2} = 15 psf (8'/2) + 12 psf (11') = 192 plf$
$w_{L_2} = 25 psf (8'/2) = 100 plf$

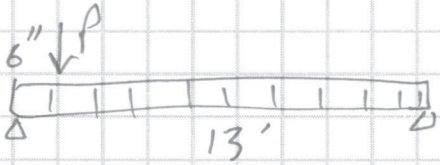
$P = 580\# DL + 300\# SL$



$1\frac{3}{4} \times 14$ LSL
minimum

(31) $w_D = 15 psf (1.33) = 20 plf$
$w_L = 40 psf (1.33) = 53.3 plf$

$P = 1350\# DL + 2250\# SL$ from (11)

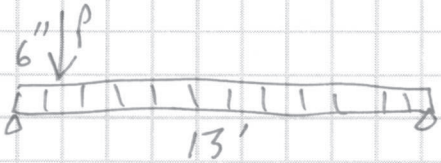


$1\frac{3}{4} \times 14$ LSL
minimum

(32) $w_D = 20 plf$
$w_L = 53.3 plf$

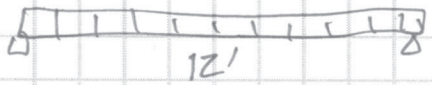
$P_1 = 1730\# DL + 2680\# SL$ from (6)

$P_2 = 3430\# DL + 5710\# SL$ from (15)



$5\frac{1}{4} \times 14$ LVL
$F_{c\perp}$ controls

(33) $w_D = 15 psf (25'/2) = 187.5 plf$
$w_L = 40 psf (25'/2) = 500 plf$



$3\frac{1}{2} \times 14$ LSL

FOR RKK
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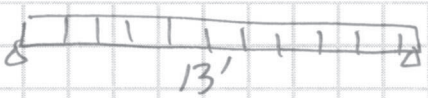
(34) $w_D = 15psf\left(1.33 + 13'/2\right) + 12psf\,(11') = 250plf$
$w_L = 53.3plf$
$w_S = 25psf\,(13'/2) = 162.5plf$



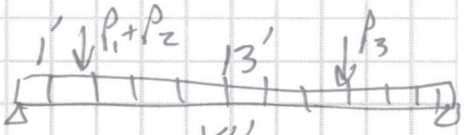
1¾ X 14 LSL

(35) $w_D = 15psf\,(26'/2) = 195plf$
$w_L = 40psf\,(26'/2) = 520plf$

$P_1 = 1630\#DL + 350\#LL + 1060\#SL$ from (34)

$P_2 = 1722\#DL + 2870\#SL$ from (7)

$P_3 = 200\#DL + 520\#LL$



5¼ X 14 LVL

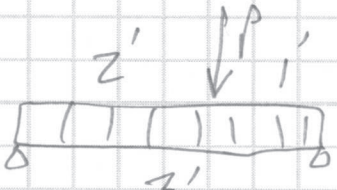
(36) $w_D = 15psf\,(13'/2) + 12psf\,(10') = 217.5plf$
$w_L = 40psf\,(13'/2) = 260plf$

3½ X 10½ GLB

(37) $w_D = 15psf\,(13'/2) + 25psf\,(15'/2) + 12psf\,(10') = 405plf$
$w_L = 40psf\,(13'/2) + 60psf\,(15'/2) = 710plf$

$P_1 = 590\#DL + 980\#SL$ from (9)

$P_2 = 5920\#DL + 4910\#LL + 3160\#SL$ from (26)



5½ X 9 GLB

FOR RKK
JOB 2434 73rd

DATE 6-10-25
BY MM

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Project Title: 2434 73rd
Engineer: Mark Myers, PE
Project ID:
Project Descr: SFR

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Description :

Wood Beam Design : 1. Rafters

Calculations per NDS 2018, IBC 2021

BEAM Size : **2x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

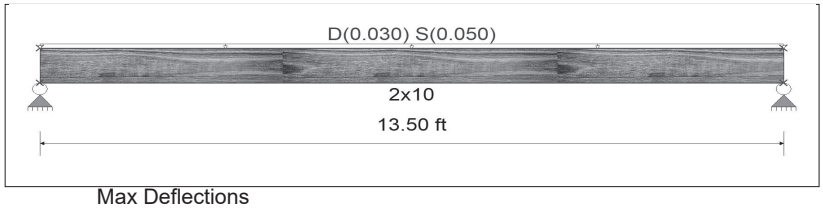
Unif Load: D = 0.0150, S = 0.0250 k/ft, Trib= 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.781** : 1
fb : Actual : 1,022.41 psi at 6.750 ft in Span # 1
Fb : Allowable : 1,309.28 psi
Load Comb : +D+S

Max fv/FvRatio = **0.250** : 1
fv : Actual : 51.76 psi at 12.735 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.20			0.34			
Right Support	0.20			0.34			



Max Deflections

Transient Downward	0.237 in	Total Downward	0.380 in
Ratio	682	Ratio	426
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 2. Ridge beam over Primary

Calculations per NDS 2018, IBC 2021

BEAM Size : **3.5x13.5, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

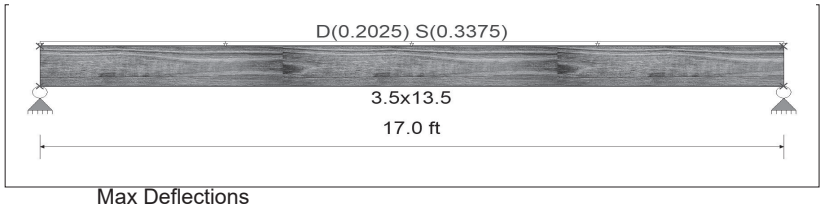
Unif Load: D = 0.2025, S = 0.3375 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.798** : 1
fb : Actual : 2,201.90 psi at 8.500 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.418** : 1
fv : Actual : 127.26 psi at 15.923 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.72			2.87			
Right Support	1.72			2.87			



Max Deflections

Transient Downward	0.494 in	Total Downward	0.790 in
Ratio	413	Ratio	258
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Project ID:
Project Descr: SFR

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Wood Beam Design : 3. Ridge beam over Primary Bath

Calculations per NDS 2018, IBC 2021

BEAM Size : **2-2x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.2		
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

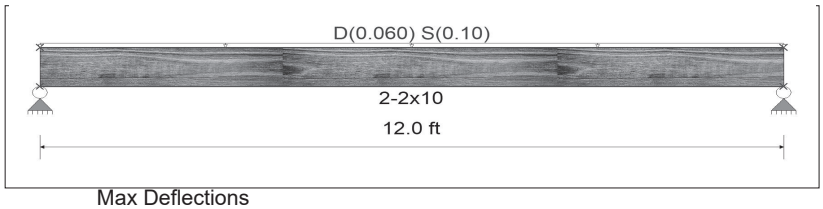
Unif Load: D = 0.060, S = 0.10 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.710** : 1
fb : Actual : 807.83 psi at 6.000 ft in Span # 1
Fb : Allowable : 1,138.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.219** : 1
fv : Actual : 45.32 psi at 11.240 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.36			0.60			
Right Support	0.36			0.60			



Max Deflections

Transient Downward	0.148 in	Total Downward	0.237 in
Ratio	971	Ratio	607
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 4. Ridge beam over Cov'd Deck

Calculations per NDS 2018, IBC 2021

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

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

Applied Loads

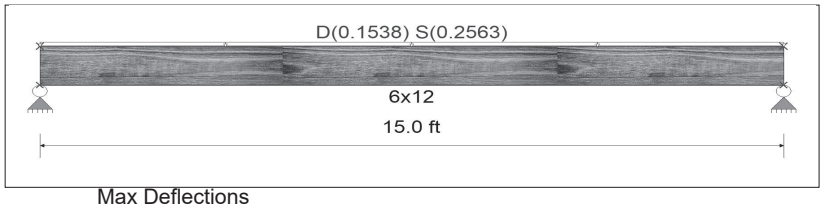
Unif Load: D = 0.1538, S = 0.2563 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.735** : 1
fb : Actual : 1,141.71 psi at 7.500 ft in Span # 1
Fb : Allowable : 1,552.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.326** : 1
fv : Actual : 63.70 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.15			1.92			
Right Support	1.15			1.92			



Max Deflections

Transient Downward	0.263 in	Total Downward	0.421 in
Ratio	684	Ratio	427
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 5. Side beam over Cov'd Deck

Calculations per NDS 2018, IBC 2021

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

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

Applied Loads

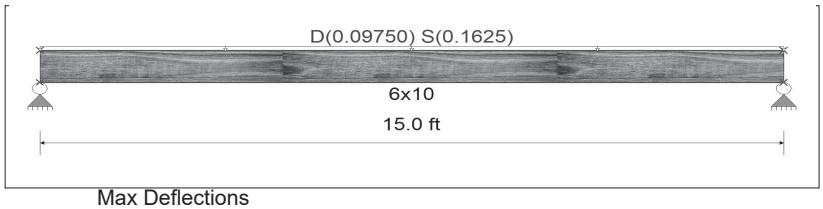
Unif Load: D = 0.09750, S = 0.1625 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.683** : 1
fb : Actual : 1,060.69 psi at 7.500 ft in Span # 1
Fb : Allowable : 1,552.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.258** : 1
fv : Actual : 50.38 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.73			1.22			
Right Support	0.73			1.22			



Max Deflections

Transient Downward	0.296 in	Total Downward	0.474 in
Ratio	608	Ratio	380
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 6. Ridge beam over Living Rm

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x15, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

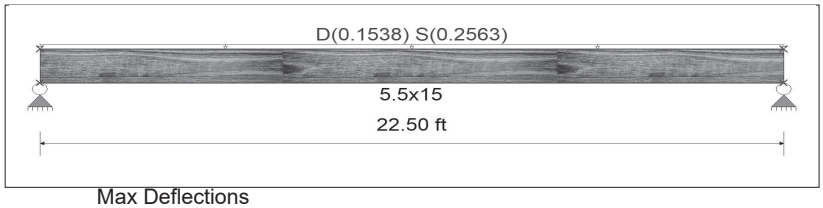
Unif Load: D = 0.1538, S = 0.2563 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.567** : 1
fb : Actual : 1,509.91 psi at 11.250 ft in Span # 1
Fb : Allowable : 2,661.67 psi
Load Comb : +D+S

Max fv/FvRatio = **0.246** : 1
fv : Actual : 74.94 psi at 21.300 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.73			2.88			
Right Support	1.73			2.88			



Max Deflections

Transient Downward	0.534 in	Total Downward	0.854 in
Ratio	505	Ratio	316
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Project Descr: SFR

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Wood Beam Design : 7. Gable Beam at Cov'd Deck

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x12, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

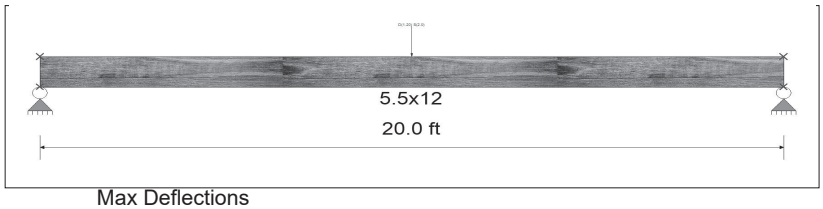
Beam self weight calculated and added to loads
1Point: D = 1.20, S = 2.0 k @ 10.0 ft

Design Summary

Max fb/Fb Ratio = **0.563** : 1
fb : Actual : 1,519.57 psi at 10.000 ft in Span # 1
Fb : Allowable : 2,696.71 psi
Load Comb : +D+S

Max fv/FvRatio = **0.129** : 1
fv : Actual : 39.31 psi at 19.067 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.74			1.00			
Right Support	0.74			1.00			



Max Deflections

Transient Downward	0.406 in	Total Downward	0.686 in
Ratio	591	Ratio	349
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 8. Header at Living Room SGD

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x13.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

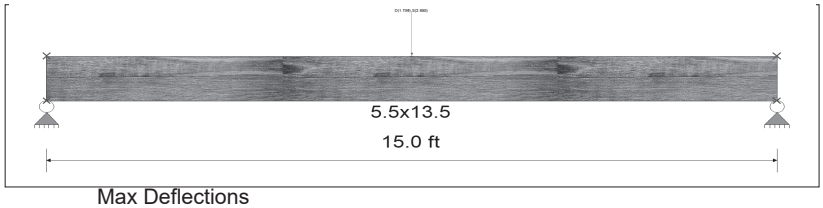
Beam self weight calculated and added to loads
1Point: D = 1.154, S = 1.922 k @ 7.50 ft
2Point: D = 1.730, S = 2.890 k @ 7.50 ft

Design Summary

Max fb/Fb Ratio = **0.777** : 1
fb : Actual : 2,105.51 psi at 7.500 ft in Span # 1
Fb : Allowable : 2,709.14 psi
Load Comb : +D+S

Max fv/FvRatio = **0.262** : 1
fv : Actual : 79.82 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.56			2.41			
Right Support	1.56			2.41			



Max Deflections

Transient Downward	0.289 in	Total Downward	0.472 in
Ratio	621	Ratio	381
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 9. Side Beam at Living Rm

Calculations per NDS 2018, IBC 2021

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

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

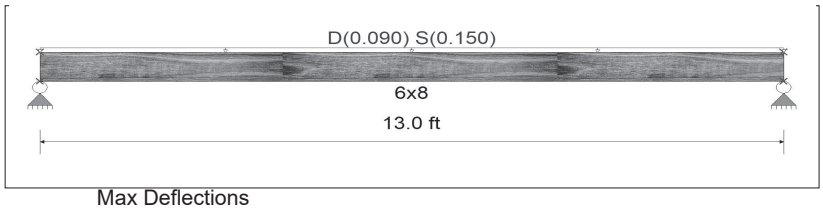
Applied Loads

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

Design Summary

Max fb/Fb Ratio = **0.767** : 1
fb : Actual : 1,179.93 psi at 6.500 ft in Span # 1
Fb : Allowable : 1,538.15 psi
Load Comb : +D+S

Max fv/FvRatio = **0.263** : 1
fv : Actual : 51.43 psi at 12.393 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.59			0.98			
Right Support	0.59			0.98			

Max Deflections

Transient Downward	0.313 in	Total Downward	0.501 in
Ratio	498	Ratio	311
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 10. Side Beam at Kitchen

Calculations per NDS 2018, IBC 2021

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

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

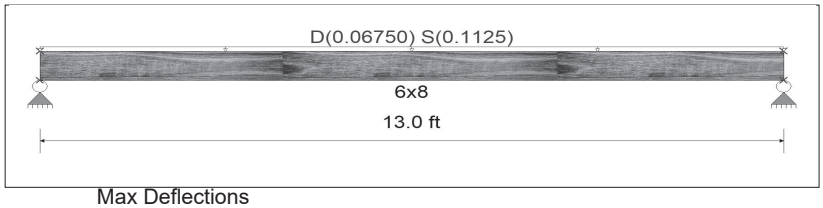
Applied Loads

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

Design Summary

Max fb/Fb Ratio = **0.575** : 1
fb : Actual : 884.95 psi at 6.500 ft in Span # 1
Fb : Allowable : 1,538.15 psi
Load Comb : +D+S

Max fv/FvRatio = **0.197** : 1
fv : Actual : 38.57 psi at 12.393 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.44			0.73			
Right Support	0.44			0.73			

Max Deflections

Transient Downward	0.235 in	Total Downward	0.376 in
Ratio	664	Ratio	415
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Project Descr: SFR

Description :

Wood Beam Design : 11. Ridge Beam at Kitchen

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x13.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

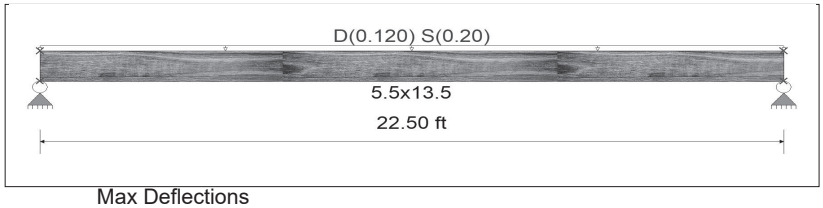
Unif Load: D = 0.120, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.545** : 1
fb : Actual : 1,454.55 psi at 11.250 ft in Span # 1
Fb : Allowable : 2,670.83 psi
Load Comb : +D+S

Max fv/FvRatio = **0.216** : 1
fv : Actual : 65.94 psi at 21.450 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.35			2.25			
Right Support	1.35			2.25			



Max Deflections

Transient Downward	0.571 in	Total Downward	0.914 in
Ratio	472	Ratio	295
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 12. Header at Dining

Calculations per NDS 2018, IBC 2021

BEAM Size : **4x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

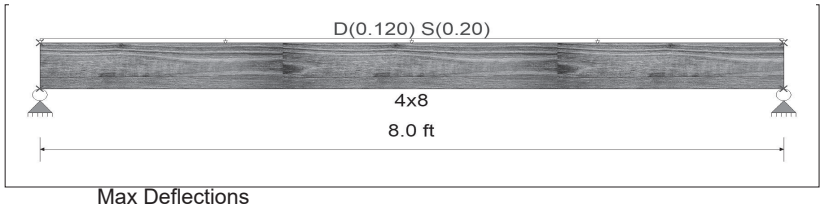
Unif Load: D = 0.120, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.754** : 1
fb : Actual : 1,001.91 psi at 4.000 ft in Span # 1
Fb : Allowable : 1,328.75 psi
Load Comb : +D+S

Max fv/FvRatio = **0.312** : 1
fv : Actual : 64.57 psi at 0.000 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.48			0.80			
Right Support	0.48			0.80			



Max Deflections

Transient Downward	0.104 in	Total Downward	0.167 in
Ratio	921	Ratio	575
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 13. Header at Primary

Calculations per NDS 2018, IBC 2021

BEAM Size : **4x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

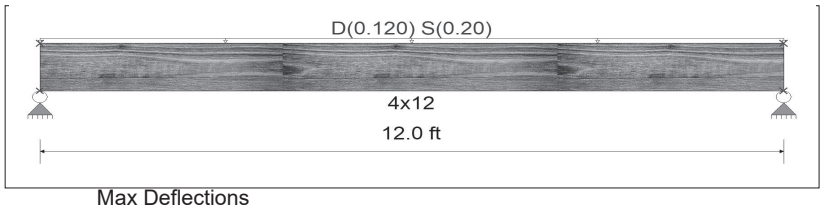
Unif Load: D = 0.120, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.849** : 1
fb : Actual : 936.23 psi at 6.000 ft in Span # 1
Fb : Allowable : 1,103.12 psi
Load Comb : +D+S

Max fv/FvRatio = **0.299** : 1
fv : Actual : 61.93 psi at 11.080 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.72			1.20			
Right Support	0.72			1.20			



Max Deflections

Transient Downward	0.141 in	Total Downward	0.226 in
Ratio	1019	Ratio	637
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 14. Header at Kitchen

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x7.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

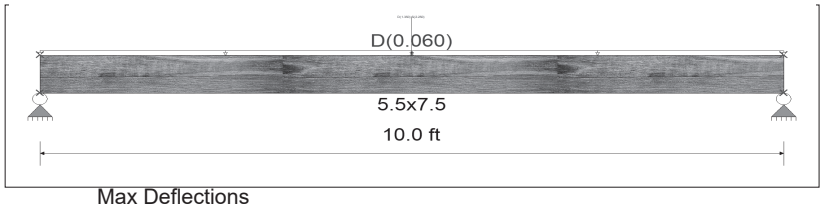
Unif Load: D = 0.060 k/ft, Trib= 1.0 ft
1Point: D = 1.350, S = 2.250 k @ 5.0 ft

Design Summary

Max fb/Fb Ratio = **0.827** : 1
fb : Actual : 2,269.09 psi at 5.000 ft in Span # 1
Fb : Allowable : 2,744.48 psi
Load Comb : +D+S

Max fv/FvRatio = **0.246** : 1
fv : Actual : 75.05 psi at 9.400 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.98			1.13			
Right Support	0.98			1.13			



Max Deflections

Transient Downward	0.234 in	Total Downward	0.413 in
Ratio	513	Ratio	290
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 15. Beam option for Box Girder Truss

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x25.5, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

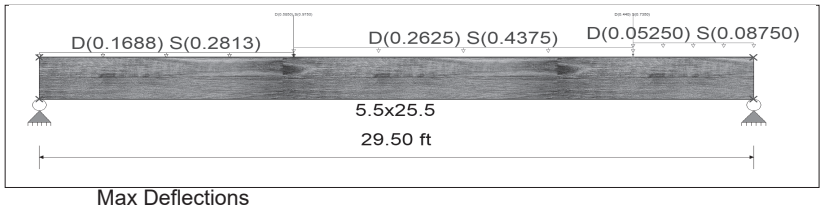
Unif Load: D = 0.1688, S = 0.2813 k/ft, 0.0 ft to 10.50 ft, Trib= 1.0 ft
Unif Load: D = 0.2625, S = 0.4375 k/ft, 10.50 to 24.50 ft, Trib= 1.0 ft
Unif Load: D = 0.05250, S = 0.08750 k/ft, 24.50 to 29.50 ft, Trib= 1.0 ft
1Point: D = 0.5850, S = 0.9750 k @ 10.50 ft
2Point: D = 0.440, S = 0.7350 k @ 24.50 ft

Design Summary

Max fb/Fb Ratio = **0.630** : 1
fb : Actual : 1,548.12 psi at 14.553 ft in Span # 1
Fb : Allowable : 2,456.67 psi
Load Comb : +D+S

Max fv/FvRatio = **0.300** : 1
fv : Actual : 91.31 psi at 27.435 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.43			5.71			
Right Support	3.31			5.52			



Max Deflections

Transient Downward	0.547 in	Total Downward	0.875 in
Ratio	647	Ratio	404
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 16. Garage Door Header

Calculations per NDS 2018, IBC 2021

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

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

Applied Loads

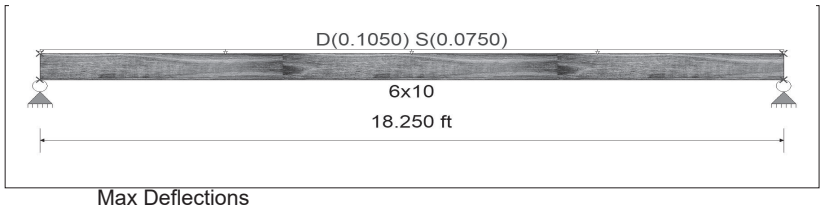
Unif Load: D = 0.1050, S = 0.0750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.714** : 1
fb : Actual : 1,087.00 psi at 9.125 ft in Span # 1
Fb : Allowable : 1,523.32 psi
Load Comb : +D+S

Max fv/FvRatio = **0.220** : 1
fv : Actual : 43.07 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.96			0.68			
Right Support	0.96			0.68			



Max Deflections

Transient Downward	0.299 in	Total Downward	0.718 in
Ratio	731	Ratio	304
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 17. Porch Ridge

Calculations per NDS 2018, IBC 2021

BEAM Size : **2x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.2		
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

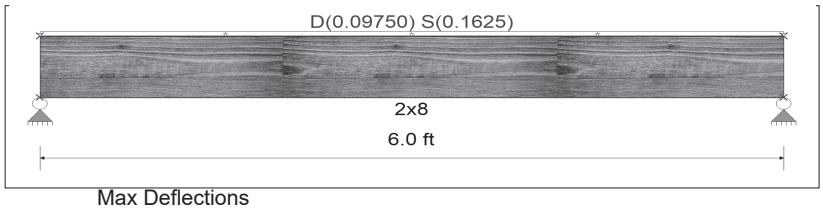
Unif Load: D = 0.09750, S = 0.1625 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.860** : 1
fb : Actual : 1,068.44 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,242.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.416** : 1
fv : Actual : 86.07 psi at 5.400 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.29			0.49			
Right Support	0.29			0.49			



Max Deflections

Transient Downward	0.063 in	Total Downward	0.100 in
Ratio	1151	Ratio	719
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 18. Porch Gable Beam

Calculations per NDS 2018, IBC 2021

BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.2		
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

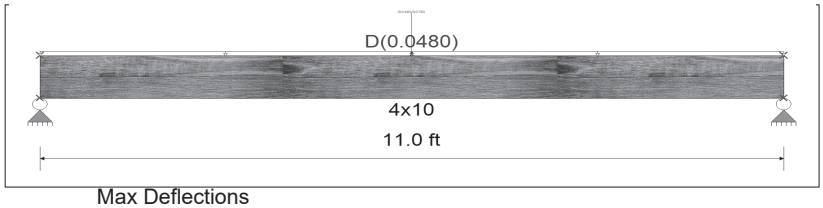
Beam self weight calculated and added to loads
Unif Load: D = 0.0480 k/ft, Trib= 1.0 ft
1Point: D = 0.440, S = 0.730 k @ 5.50 ft

Design Summary

Max fb/Fb Ratio = **0.802** : 1
fb : Actual : 973.64 psi at 5.500 ft in Span # 1
Fb : Allowable : 1,213.47 psi
Load Comb : +D+S

Max fv/FvRatio = **0.189** : 1
fv : Actual : 39.16 psi at 0.000 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.52			0.37			
Right Support	0.52			0.37			



Max Deflections

Transient Downward	0.095 in	Total Downward	0.202 in
Ratio	1386	Ratio	653
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 19. Header at Office

Calculations per NDS 2018, IBC 2021

BEAM Size : **4x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch					Wood Grade :	No.2		
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	31.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

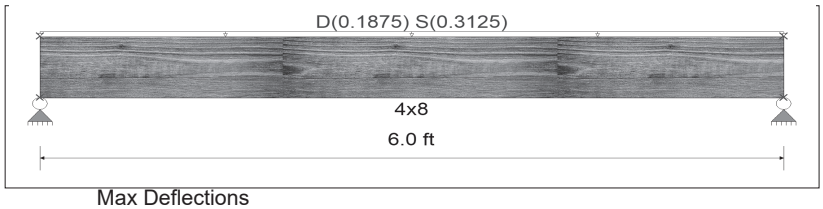
Unif Load: D = 0.1875, S = 0.3125 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.661** : 1
fb : Actual : 880.58 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,333.02 psi
Load Comb : +D+S

Max fv/FvRatio = **0.343** : 1
fv : Actual : 70.94 psi at 5.400 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.56			0.94			
Right Support	0.56			0.94			



Max Deflections

Transient Downward	0.052 in	Total Downward	0.082 in
Ratio	1397	Ratio	873
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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.

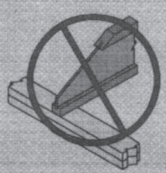
General Notes

- 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.

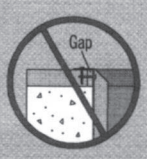
Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use TJ Pro™ Ratings included in ForteWEB® and our span table web application.

These Conditions Are NOT Permitted:

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



DO NOT bevel cut joist beyond inside face of wall.



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

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Project ID:
Project Descr: SFR

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Description :

Wood Beam Design : 21. Deck Joists

Calculations per NDS 2018, IBC 2021

BEAM Size : **2x12, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	Hem-Fir				Wood Grade :	No.2			
Fb - Tension	850.0 psi	Fc - Prll	1,300.0 psi	Fv	150.0 psi	Ebend- xx	1,300.0 ksi	Density	26.840 pcf
Fb - Compr	850.0 psi	Fc - Perp	405.0 psi	Ft	525.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

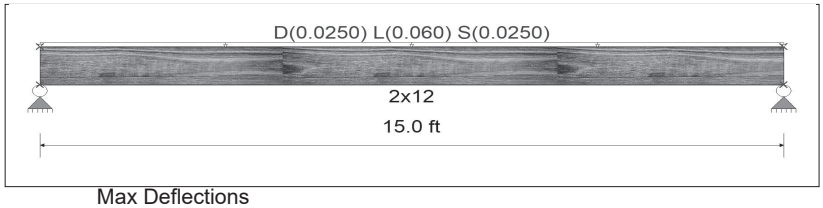
Unif Load: D = 0.0250, L = 0.060, S = 0.0250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.928** : 1
fb : Actual : 906.67 psi at 7.500 ft in Span # 1
Fb : Allowable : 977.50 psi
Load Comb : +D+L

Max fv/FvRatio = **0.332** : 1
fv : Actual : 49.87 psi at 0.000 ft in Span # 1
Fv : Allowable : 150.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.19		0.45	0.19			
Right Support	0.19		0.45	0.19			



Max Deflections

Transient Downward	0.297 in	Total Downward	0.439 in
Ratio	606	Ratio	409
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 22. Beam at Open Deck

Calculations per NDS 2018, IBC 2021

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

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

Applied Loads

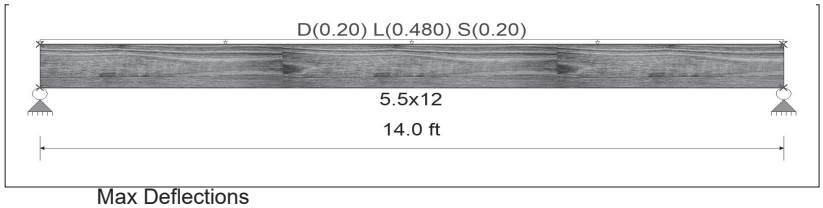
Unif Load: D = 0.20, L = 0.480, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.631** : 1
fb : Actual : 1,514.55 psi at 7.000 ft in Span # 1
Fb : Allowable : 2,400.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.351** : 1
fv : Actual : 93.04 psi at 13.020 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.40		3.36	1.40			
Right Support	1.40		3.36	1.40			



Max Deflections

Transient Downward	0.293 in	Total Downward	0.433 in
Ratio	574	Ratio	388
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 23. Beam at Cov'd Deck

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x15, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

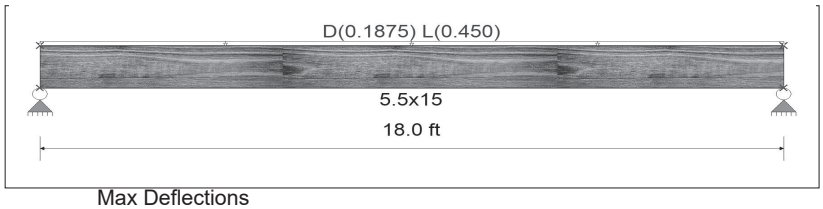
Applied Loads

Unif Load: D = 0.1875, L = 0.450 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.635** : 1
fb : Actual : 1,502.18 psi at 9.000 ft in Span # 1
Fb : Allowable : 2,366.73 psi
Load Comb : +D+L

Max fv/FvRatio = **0.341** : 1
fv : Actual : 90.41 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.69		4.05				
Right Support	1.69		4.05				

Max Deflections

Transient Downward	0.384 in	Total Downward	0.544 in
Ratio	562	Ratio	397
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 24. Floor Beam at Living Rm Gable Wall

Calculations per NDS 2018, IBC 2021

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

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

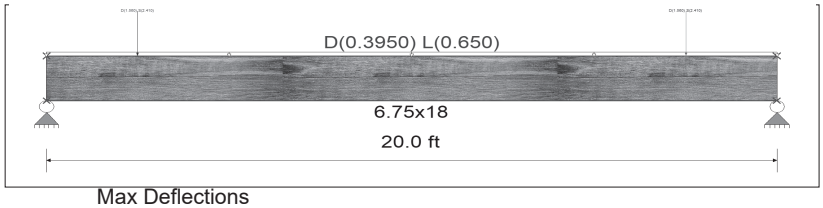
Applied Loads

Unif Load: D = 0.3950, L = 0.650 k/ft, Trib= 1.0 ft
1Point: D = 1.560, S = 2.410 k @ 2.50 ft
2Point: D = 1.560, S = 2.410 k @ 17.50 ft

Design Summary

Max fb/Fb Ratio = **0.820** : 1
fb : Actual : 1,848.56 psi at 10.000 ft in Span # 1
Fb : Allowable : 2,252.99 psi
Load Comb : +D+L

Max fv/FvRatio = **0.488** : 1
fv : Actual : 129.35 psi at 18.533 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	5.51		6.50	2.41			
Right Support	5.51		6.50	2.41			

Max Deflections

Transient Downward	0.398 in	Total Downward	0.697 in
Ratio	602	Ratio	344
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 24. Floor Beam at Living Rm Gable Wall

Calculations per NDS 2018, IBC 2021

BEAM Size : **7x16, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

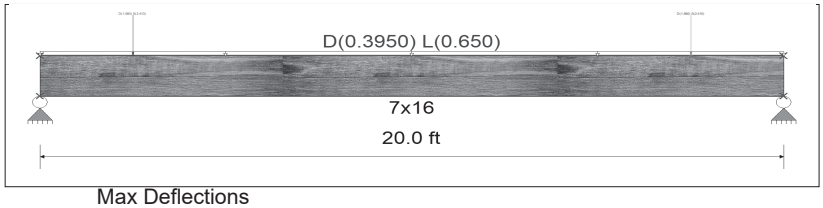
Unif Load: D = 0.3950, L = 0.650 k/ft, Trib= 1.0 ft
1Point: D = 1.560, S = 2.410 k @ 2.50 ft
2Point: D = 1.560, S = 2.410 k @ 17.50 ft

Design Summary

Max fb/Fb Ratio = **0.803** : 1
fb : Actual : 2,256.03 psi at 10.000 ft in Span # 1
Fb : Allowable : 2,808.86 psi
Load Comb : +D+L

Max fv/FvRatio = **0.494** : 1
fv : Actual : 143.12 psi at 18.733 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	5.51		6.50	2.41			
Right Support	5.51		6.50	2.41			



Max Deflections

Transient Downward	0.492 in	Total Downward	0.861 in
Ratio	487	Ratio	278
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 25. Floor Beam supporting Beam 24

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x24, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

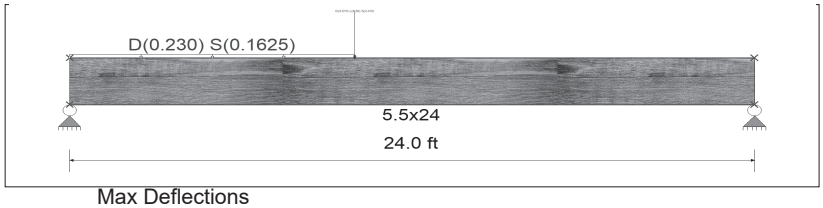
Beam self weight calculated and added to loads
Unif Load: D = 0.230, S = 0.1625 k/ft, 0.0 ft to 10.0 ft, Trib= 1.0 ft
1Point: D = 5.510, L = 6.50, S = 2.410 k @ 10.0 ft

Design Summary

Max fb/Fb Ratio = **0.816** : 1
fb : Actual : 1,790.21 psi at 10.000 ft in Span # 1
Fb : Allowable : 2,194.03 psi
Load Comb : +D+L

Max fv/FvRatio = **0.371** : 1
fv : Actual : 98.33 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	5.38		3.79	2.69			
Right Support	3.12		2.71	1.34			



Max Deflections

Transient Downward	0.275 in	Total Downward	0.621 in
Ratio	1047	Ratio	464
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam Design : 26. Floor Beam supporting Beams 24 & 22

Calculations per NDS 2018, IBC 2021

BEAM Size : **6.75x24, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

Applied Loads

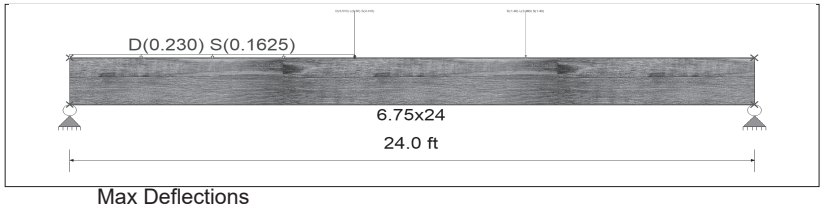
Beam self weight calculated and added to loads
Unif Load: D = 0.230, S = 0.1625 k/ft, 0.0 ft to 10.0 ft, Trib= 1.0 ft
1Point: D = 5.510, L = 6.50, S = 2.410 k @ 10.0 ft
2Point: D = 1.40, L = 3.360, S = 1.40 k @ 16.0 ft

Design Summary

Max fb/Fb Ratio = **0.819** : 1
fb : Actual : 1,760.95 psi at 10.000 ft in Span # 1
Fb : Allowable : 2,149.55 psi
Load Comb : +D+L

Max fv/FvRatio = **0.360** : 1
fv : Actual : 95.41 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	5.92		4.91	3.16			
Right Support	4.13		4.95	2.28			



Max Deflections

Transient Downward	0.325 in	Total Downward	0.659 in
Ratio	884	Ratio	437
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 27. Floor Beam at Kitchenb Gable Wall

Calculations per NDS 2018, IBC 2021

BEAM Size : **3.5x14.0, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist					Wood Grade :	Parallam PSL 2.0E		
Fb - Tension	2900 psi	Fc - Prll	2900 psi	Fv	290 psi	Ebend- xx	2000 ksi	Density	45.07 pcf
Fb - Compr	2900 psi	Fc - Perp	750 psi	Ft	2025 psi	Eminbend - xx	1016.535 ksi		

Applied Loads

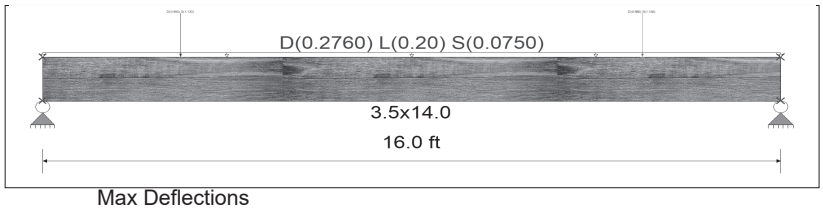
Unif Load: D = 0.2760, L = 0.20, S = 0.0750 k/ft, Trib= 1.0 ft
1Point: D = 0.980, S = 1.130 k @ 3.0 ft
2Point: D = 0.980, S = 1.130 k @ 13.0 ft

Design Summary

Max fb/Fb Ratio = **0.859** : 1
fb : Actual : 2,195.11 psi at 8.000 ft in Span # 1
Fb : Allowable : 2,555.40 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.472** : 1
fv : Actual : 157.51 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	3.19		1.60	1.73			
Right Support	3.19		1.60	1.73			



Max Deflections

Transient Downward	0.185 in	Total Downward	0.628 in
Ratio	1036	Ratio	305
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Wood Beam

Project File: 2434 73rd Beam Calcs.ec6

DESCRIPTION: 28. Cantilever Deck Beam supporting Beam 22

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : NDS 2018
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

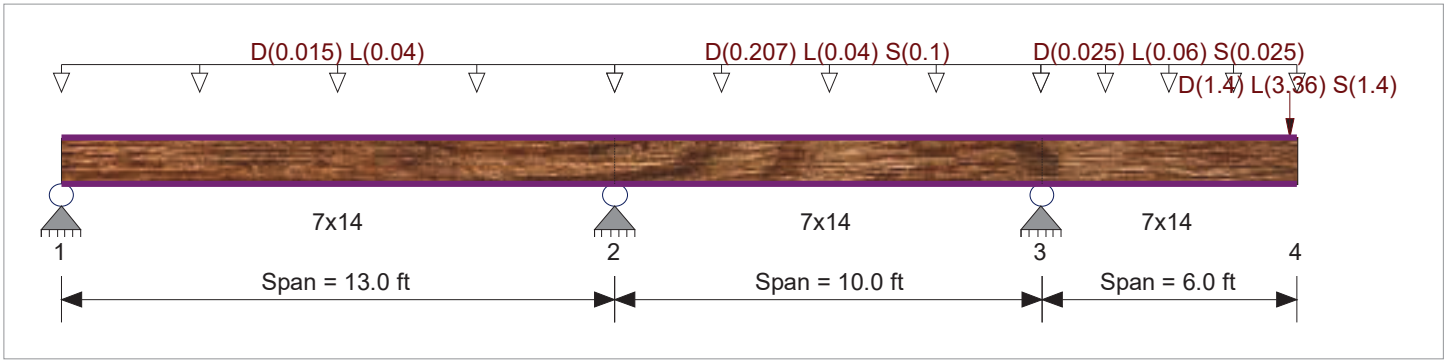
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

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	2,000.0 ksi
Eminbend - xx	1,016.54 ksi
Density	45.070 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.0150, L = 0.040 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.2070, L = 0.040, S = 0.10 , Tributary Width = 1.0 ft
Load for Span Number 3
Uniform Load : D = 0.0250, L = 0.060, S = 0.0250 , Tributary Width = 1.0 ft
Point Load : D = 1.40, L = 3.360, S = 1.40 k @ 5.830 ft, (Load from Beam 22)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.539 : 1	Maximum Shear Stress Ratio	=	0.273 : 1
Section used for this span		7x14	Section used for this span		7x14
fb: Actual	=	1,536.60 psi	fv: Actual	=	79.15 psi
F'b	=	2,850.80 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	10.000 ft	Location of maximum on span	=	10.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.327	in	Ratio =	440 >=360	Span: 3 : L Only
Max Upward Transient Deflection	-0.058	in	Ratio =	2078 >=360	Span: 2 : L Only
Max Downward Total Deflection	0.456	in	Ratio =	316 >=240	Span: 3 : +D+0.750L+0.750S
Max Upward Total Deflection	-0.072	in	Ratio =	1662 >=240	Span: 2 : +D+0.750L+0.750S

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.693	0.171	10.561	
Max Upward from Load Combinations	0.693	0.102	10.561	
Max Upward from Load Cases	0.552	0.171	6.367	
Max Downward from all Load Conditions (Resisting Uplift)		-2.279		
Max Downward from Load Combinations (Resisting Uplift)		-2.108		

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DESCRIPTION: 28. Cantilever Deck Beam supporting Beam 22

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Downward from Load Cases (Resisting Uplift)		-2.279		
D Only	0.141	0.171	3.503	
+D+L	0.693	-2.108	9.870	
+D+S	0.243	-0.425	6.547	
+D+0.750L	0.555	-1.538	8.278	
+D+0.750L+0.750S	0.632	-1.986	10.561	
+0.60D	0.085	0.102	2.102	
L Only	0.552	-2.279	6.367	
S Only	0.102	-0.596	3.044	

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Wood Beam

Project File: 2434 73rd Beam Calcs.ec6

DESCRIPTION: 29. Header below Beam 28

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : NDS 2018
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

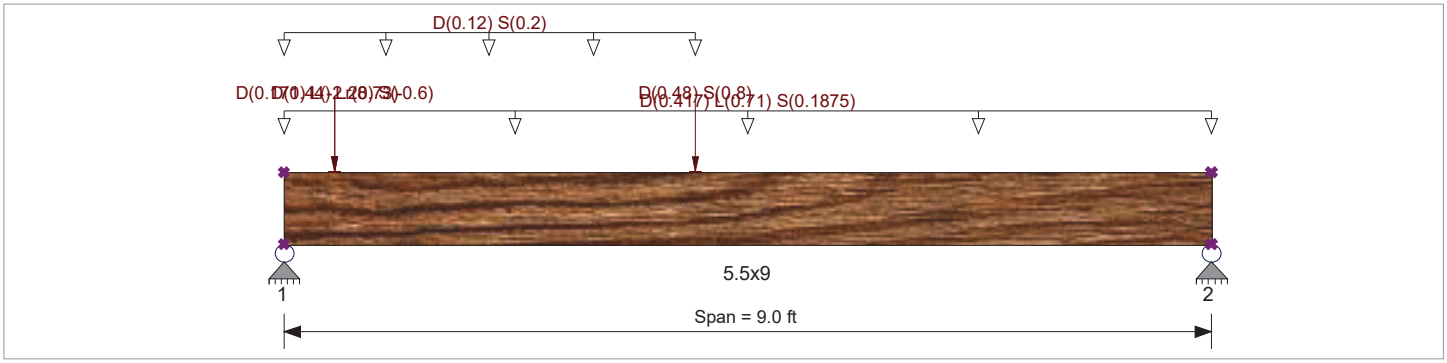
Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Completely Unbraced

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

E : Modulus of Elasticity

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.4170, L = 0.710, S = 0.1875 , Tributary Width = 1.0 ft
Uniform Load : D = 0.120, S = 0.20 k/ft, Extent = 0.0 -->> 4.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.1710, L = -2.280, S = -0.60 k @ 0.50 ft
Point Load : D = 0.440, Lr = 0.730 k @ 0.50 ft
Point Load : D = 0.480, S = 0.80 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.843 : 1	Maximum Shear Stress Ratio	=	0.556 : 1
Section used for this span		5.5x9	Section used for this span		5.5x9
fb: Actual	=	2,013.18 psi	fv: Actual	=	147.34 psi
F'b	=	2,387.10 psi	F'v	=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.303 ft	Location of maximum on span	=	0.755 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.159 in	Ratio =	680	>=480	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<480	n/a
Max Downward Total Deflection	0.332 in	Ratio =	325	>=240	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.883	5.540
Max Upward from Load Combinations	4.883	5.540
Max Upward from Load Cases	3.094	3.068
D Only	3.094	2.230
+D+L	4.135	5.299
+D+Lr	3.783	2.271
+D+S	4.437	3.574

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Wood Beam

DESCRIPTION: 29. Header below Beam 28

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L	4.392	4.562
+D+0.750L+0.750S	4.883	5.540
+0.60D	1.856	1.338
Lr Only	0.689	0.041
L Only	1.042	3.068
S Only	1.344	1.344

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Wood Beam

Project File: 2434 73rd Beam Calcs.ec6

DESCRIPTION: 30. Floor beam at Kitchen/Pantry

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : NDS 2018
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

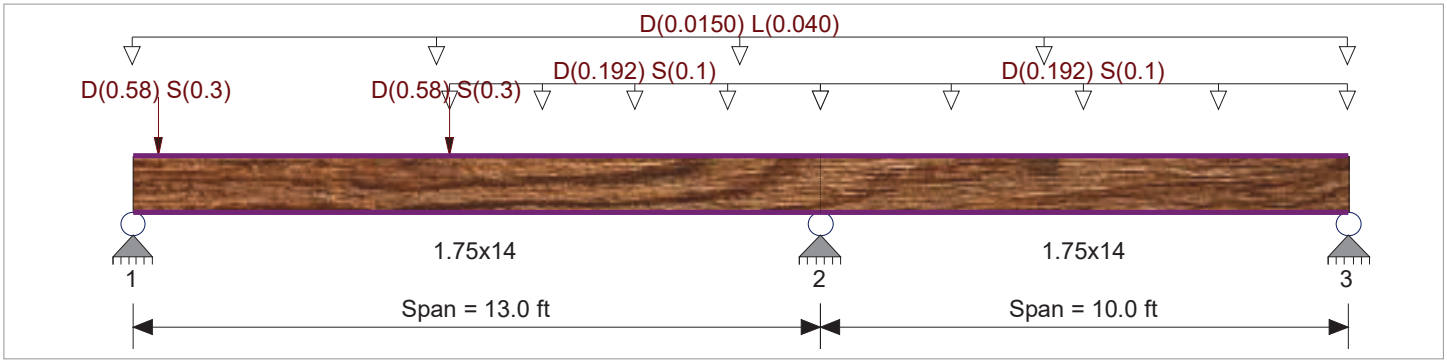
Wood Species : iLevel Truss Joist
Wood Grade : TimberStrand LSL 1.55E

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

Fb +	2325 psi
Fb -	2325 psi
Fc - Prll	2050 psi
Fc - Perp	800 psi
Fv	310 psi
Ft	1070 psi

E : Modulus of Elasticity

Ebend- xx	1550 ksi
Eminbend - xx	787.815 ksi
Density	45.01 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, L = 0.040 k/ft
Load for Span Number 1
Uniform Load : D = 0.1920, S = 0.10 k/ft, Extent = 6.0 -->> 13.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.580, S = 0.30 k @ 0.50 ft
Point Load : D = 0.580, S = 0.30 k @ 6.0 ft
Load for Span Number 2
Uniform Load : D = 0.1920, S = 0.10 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.430 : 1	Maximum Shear Stress Ratio	=	0.363 : 1
Section used for this span		1.75x14	Section used for this span		1.75x14
fb: Actual	=	1,132.53 psi	fv: Actual	=	129.38 psi
F'b	=	2,636.10 psi	F'v	=	356.50 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	11.838 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.051 in	Ratio = 3085	>=480	Span: 1 : S Only
Max Upward Transient Deflection	-0.002 in	Ratio = 48387	>=480	Span: 2 : S Only
Max Downward Total Deflection	0.160 in	Ratio = 977	>=240	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	-0.008 in	Ratio = 15301	>=240	Span: 2 : +D+0.750L+0.750S

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	1.588	4.575	1.020
Max Upward from Load Combinations	1.588	4.575	1.020
Max Upward from Load Cases	1.054	3.038	0.677
D Only	1.054	3.038	0.677

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Wood Beam

DESCRIPTION: 30. Floor beam at Kitchen/Pantry

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+L	1.261	3.621	0.808
+D+S	1.560	4.505	1.004
+D+0.750L	1.209	3.475	0.775
+D+0.750L+0.750S	1.588	4.575	1.020
+0.60D	0.632	1.823	0.406
L Only	0.207	0.583	0.131
S Only	0.506	1.467	0.327

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Description :

Wood Beam Design : 31. Floor beam transfering Kitchen Ridge Ld

Calculations per NDS 2018, IBC 2021

BEAM Size : **1.75x14, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist Wood Grade : TimberStrand LSL 1.55E

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

Applied Loads

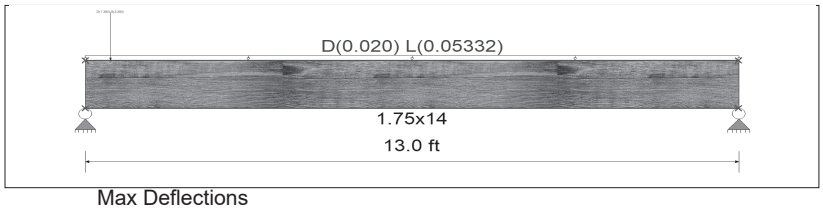
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.333 ft
1Point: D = 1.350, S = 2.250 k @ 0.50 ft

Design Summary

Max fb/Fb Ratio = **0.174** : 1
fb : Actual : 399.81 psi at 5.807 ft in Span # 1
Fb : Allowable : 2,292.26 psi
Load Comb : +D+L

Max fv/FvRatio = **0.088** : 1
fv : Actual : 27.30 psi at 11.873 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.43		0.35	2.16			
Right Support	0.18		0.35	0.09			



Max Deflections

Transient Downward	0.056 in	Total Downward	0.108 in
Ratio	2808	Ratio	1445
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 32. Floor beam transfering Living Rm Ridge & Box Girder

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.25x14.0, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2600 psi	Fc - Prll	2510 psi	Fv	285 psi	Ebend- xx	1900 ksi	Density	42.01 pcf
Fb - Compr	2600 psi	Fc - Perp	750 psi	Ft	1555 psi	Eminbend - xx	965.71 ksi		

Applied Loads

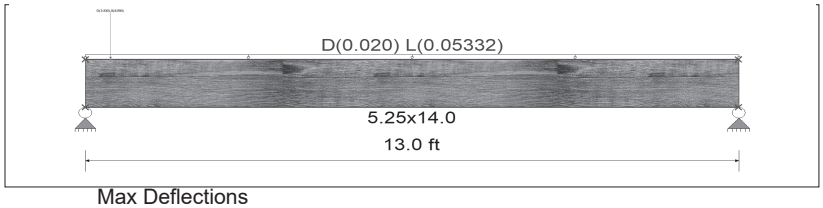
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 1.333 ft
1Point: D = 1.730, S = 2.880 k @ 0.50 ft
2Point: D = 3.430, S = 5.710 k @ 0.50 ft

Design Summary

Max fb/Fb Ratio = **0.159** : 1
fb : Actual : 466.35 psi at 0.520 ft in Span # 1
Fb : Allowable : 2,939.27 psi
Load Comb : +D+S

Max fv/FvRatio = **0.048** : 1
fv : Actual : 15.69 psi at 11.873 ft in Span # 1
Fv : Allowable : 327.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	5.09		0.35	8.26			
Right Support	0.33		0.35	0.33			



Max Deflections

Transient Downward	0.035 in	Total Downward	0.065 in
Ratio	4395	Ratio	2415
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 33. Floor Beam supporting Stairwell rim beam

Calculations per NDS 2018, IBC 2021

BEAM Size : **3.5x14, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist					Wood Grade :	TimberStrand LSL 1.55E		
Fb - Tension	2325 psi	Fc - Prll	2050 psi	Fv	310 psi	Ebend- xx	1550 ksi	Density	45.01 pcf
Fb - Compr	2325 psi	Fc - Perp	800 psi	Ft	1070 psi	Eminbend - xx	787.815 ksi		

Applied Loads

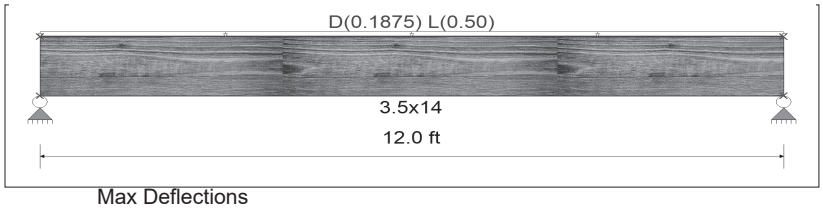
Unif Load: D = 0.1875, L = 0.50 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.567** : 1
fb : Actual : 1,298.83 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,292.26 psi
Load Comb : +D+L

Max fv/FvRatio = **0.329** : 1
fv : Actual : 101.86 psi at 10.840 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.13		3.00				
Right Support	1.13		3.00				



Max Deflections

Transient Downward	0.189 in	Total Downward	0.260 in
Ratio	761	Ratio	553
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 34. Floor beam supporting wall of Living/Primary

Calculations per NDS 2018, IBC 2021

BEAM Size : **1.75x14, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist					Wood Grade :	TimberStrand LSL 1.55E		
Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	45.010 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

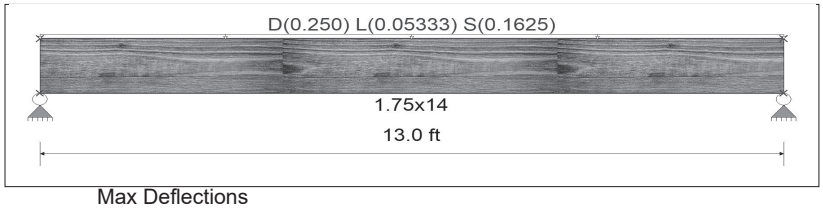
Unif Load: D = 0.250, L = 0.05333, S = 0.1625 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.694** : 1
fb : Actual : 1,829.19 psi at 6.500 ft in Span # 1
Fb : Allowable : 2,636.10 psi
Load Comb : +D+S

Max fv/FvRatio = **0.381** : 1
fv : Actual : 135.70 psi at 0.000 ft in Span # 1
Fv : Allowable : 356.50 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.63		0.35	1.06			
Right Support	1.63		0.35	1.06			



Max Deflections

Transient Downward	0.169 in	Total Downward	0.430 in
Ratio	921	Ratio	363
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 35. Floor beam at Primary

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.25x14.0, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2600 psi	Fc - Prll	2510 psi	Fv	285 psi	Ebend- xx	1900 ksi	Density	42.01 pcf
Fb - Compr	2600 psi	Fc - Perp	750 psi	Ft	1555 psi	Eminbend - xx	965.71 ksi		

Applied Loads

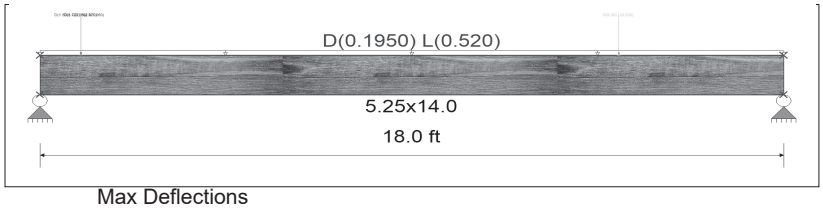
Unif Load: D = 0.1950, L = 0.520 k/ft, Trib= 1.0 ft
1Point: D = 1.630, L = 0.350, S = 1.060 k @ 1.0 ft
2Point: D = 1.722, S = 2.870 k @ 1.0 ft
3Point: D = 0.20, L = 0.520 k @ 14.0 ft

Design Summary

Max fb/Fb Ratio = **0.883** : 1
fb : Actual : 2,256.56 psi at 8.940 ft in Span # 1
Fb : Allowable : 2,555.89 psi
Load Comb : +D+L

Max fv/FvRatio = **0.457** : 1
fv : Actual : 130.32 psi at 16.860 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	4.97		5.13	3.71			
Right Support	2.10		5.10	0.22			



Max Deflections

Transient Downward	0.577 in	Total Downward	0.843 in
Ratio	374	Ratio	256
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : 36. Header at Media Rm SGD

Calculations per NDS 2018, IBC 2021

BEAM Size : **3.5x10.5, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V4

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

Applied Loads

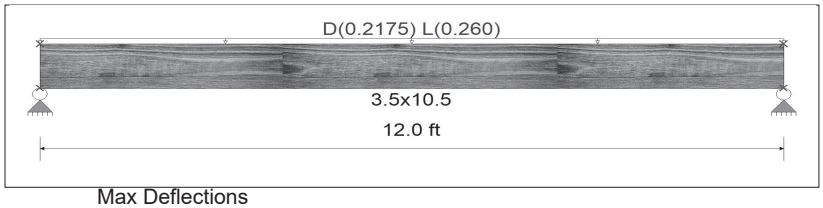
Unif Load: D = 0.2175, L = 0.260 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.687** : 1
fb : Actual : 1,603.73 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,334.30 psi
Load Comb : +D+L

Max fv/FvRatio = **0.379** : 1
fv : Actual : 100.57 psi at 11.160 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.31		1.56				
Right Support	1.31		1.56				



Max Deflections

Transient Downward	0.201 in	Total Downward	0.369 in
Ratio	717	Ratio	390
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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Multiple Simple Beam

Project File: 2434 73rd Beam Calcs.ec6

Wood Beam Design : 37. Header supporting beam 26

Calculations per NDS 2018, IBC 2021

BEAM Size : **5.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

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

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

Applied Loads

Unif Load: D = 0.4050, L = 0.710 k/ft, Trib= 1.0 ft
1Point: D = 0.590, S = 0.980 k @ 2.0 ft
2Point: D = 5.920, L = 4.910, S = 3.160 k @ 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.589** : 1
fb : Actual : 1,410.64 psi at 2.000 ft in Span # 1
Fb : Allowable : 2,395.56 psi
Load Comb : +D+L

Max fv/FvRatio = **0.967** : 1
fv : Actual : 256.39 psi at 2.260 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)

	D	Lr	L	S	W	E	H
Left Support	2.78		2.70	1.38			
Right Support	4.95		4.34	2.76			

Max Deflections

Transient Downward	0.009 in	Total Downward	0.021 in
Ratio	3997	Ratio	1681
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

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PROJECT : **2434 73rd AVE SE**

Maximum Load For 6x6 DF#1 Wood Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 11 \cdot ft$

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 833.33\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.63$$

$F'_c := C_p \cdot F''_c$ $F'_c = 625\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 18906\ lb$ (Maximum post Capacity)

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

Maximum Load For 6x6 HF#2 Treated Post

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 11 \cdot ft$

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 544.27\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.75$$

$F'_c := C_p \cdot F''_c$ $F'_c = 343\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 10377\ lb$ (Maximum post Capacity)

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

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PROJECT : **2434 73rd AVE SE**

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

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 11 \cdot ft$

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 625.00\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.56$$

$F'_c := C_p \cdot F''_c$ $F'_c = 496\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 12288\ lb$ (Maximum post Capacity)

(3)2x6 Wood Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

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

$psf := \frac{psi}{144}$ $plf := psf \cdot ft$ $lb := plf \cdot ft$ $H := 11 \cdot ft$

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 625.00\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.56$$

$F'_c := C_p \cdot F''_c$ $F'_c = 496\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 8192\ lb$ (Maximum post Capacity)

(2)2x6 Wood Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 378.09\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.38$$

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

(3)2x4 Wood Post Properties

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

$h := 3.5 \cdot in$

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

$A := t \cdot h$ $A = 15.8\ in^2$

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

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

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

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

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

$E := 1200000 \cdot psi$

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

Axial Load Capacity:
Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 378.09\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.38$$

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

(2)2x4 Wood Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

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PROJECT : **2434 73rd AVE SE**

Maximum Load For 4x4 HF#2 Treated Post

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

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

$E := 1235000 \cdot psi$

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

Axial Load Capacity:

Slenderness Ratio (SL)

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

$F_{CE} := \frac{K_{CE} \cdot E'}{SL^2} = 389.11\ psi$

$$C_p := \left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot C} \right)^2 - \frac{\frac{F_{CE}}{F''_c}}{C}} \right) \cdot K_f = 0.34$$

$F'_c := C_p \cdot F''_c$ $F'_c = 353\ psi$ $P_{max} := F'_c \cdot A$ $P_{max} = 4323\ lb$ (Maximum post Capacity)

6x6 Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

$A := t \cdot h$ $A = 12.3\ in^2$

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

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

Mark Myers, PE 07/01/2025