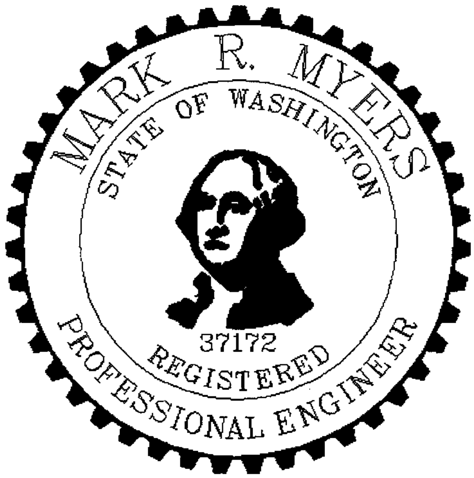
MYERS ENGINEERING

LATERAL ANALYSIS & GRAVITY CALCULATIONS



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Date: 2021.04.01 13:18:31 -07'00'

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

Project: American Classic Homes
80xx Southeast 20th Street
Mercer Island, WA

April 1, 2021

2018 INTERNATIONAL BUILDING CODE
100 MPH WIND, EXPOSURE C, K_{zt} = 1.00
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

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3206 50th Street Ct NW, Ste 210-B
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PROJECT :**80xx SE 20th Street**

Phone: 253-858-3248
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DESIGN LOADS:

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

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

WOODS : WOOD TYPE:

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

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

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

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

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

TRUSSES:

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

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

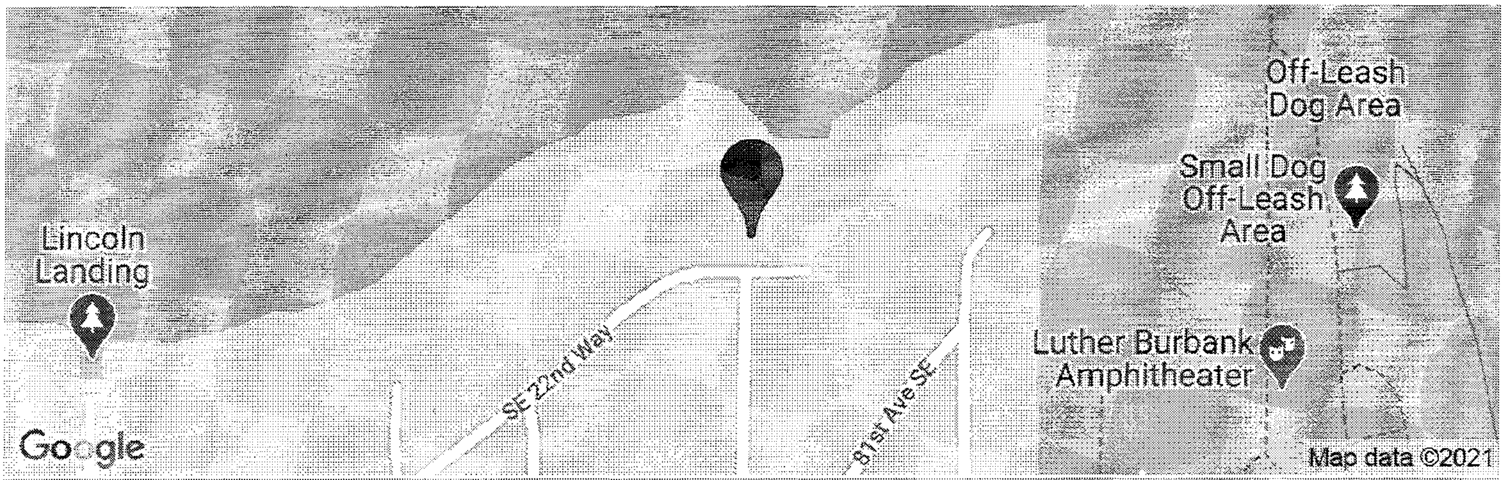
ENGINEERED I-JOISTS

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

OSHPD

80xx SE 20th St

Latitude, Longitude: 47.593, -122.2316



Date	3/2/2021, 12:00:04 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.379	MCE_R ground motion. (for 0.2 second period)
S_1	0.481	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.379	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8 0.866	Site-modified spectral acceleration value
S_{DS}	0.92	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.59	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.649	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.379	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.528	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.077	Factored deterministic acceleration value. (0.2 second)
S1RT	0.481	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.536	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.273	Factored deterministic acceleration value. (1.0 second)
PGAd	1.071	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.903	Mapped value of the risk coefficient at short periods
C_{R1}	0.896	Mapped value of the risk coefficient at a period of 1 s

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

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

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

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

Seismic Design Data:

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

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

$S_s := 1.379$ $\quad S_1 := 0.481$ $\quad S_{ms} := 1.379$ $\quad S_{m1} := 0.866$

Equation 11.4-3 $\quad S_{DS} := \frac{2}{3} \cdot S_{ms} = 0.92$ $\quad$ Equation 11.4-4 $\quad S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.58$

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

Roof Slope Adjustment Factor: $\quad S_a := \frac{1}{\cos\left(\operatorname{atan}\left(\frac{5}{12}\right)\right)} = 1.08$

Plan Area for Each Level:

$A_1 := 1840 ft^2 \cdot S_a$ (Upper Roof) $\quad A_{2a} := 1612 ft^2$ (Upper Floor) $\quad A_{2b} := 1505 ft^2 \cdot S_a$ (Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(36ft) + 2(54ft)$ (Upper Floor) $\quad P_2 := 2(54ft) + 2(54ft)$ (Main Floor)

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

Weight of Structure at Each Level:

Story Weight at Upper Floor:

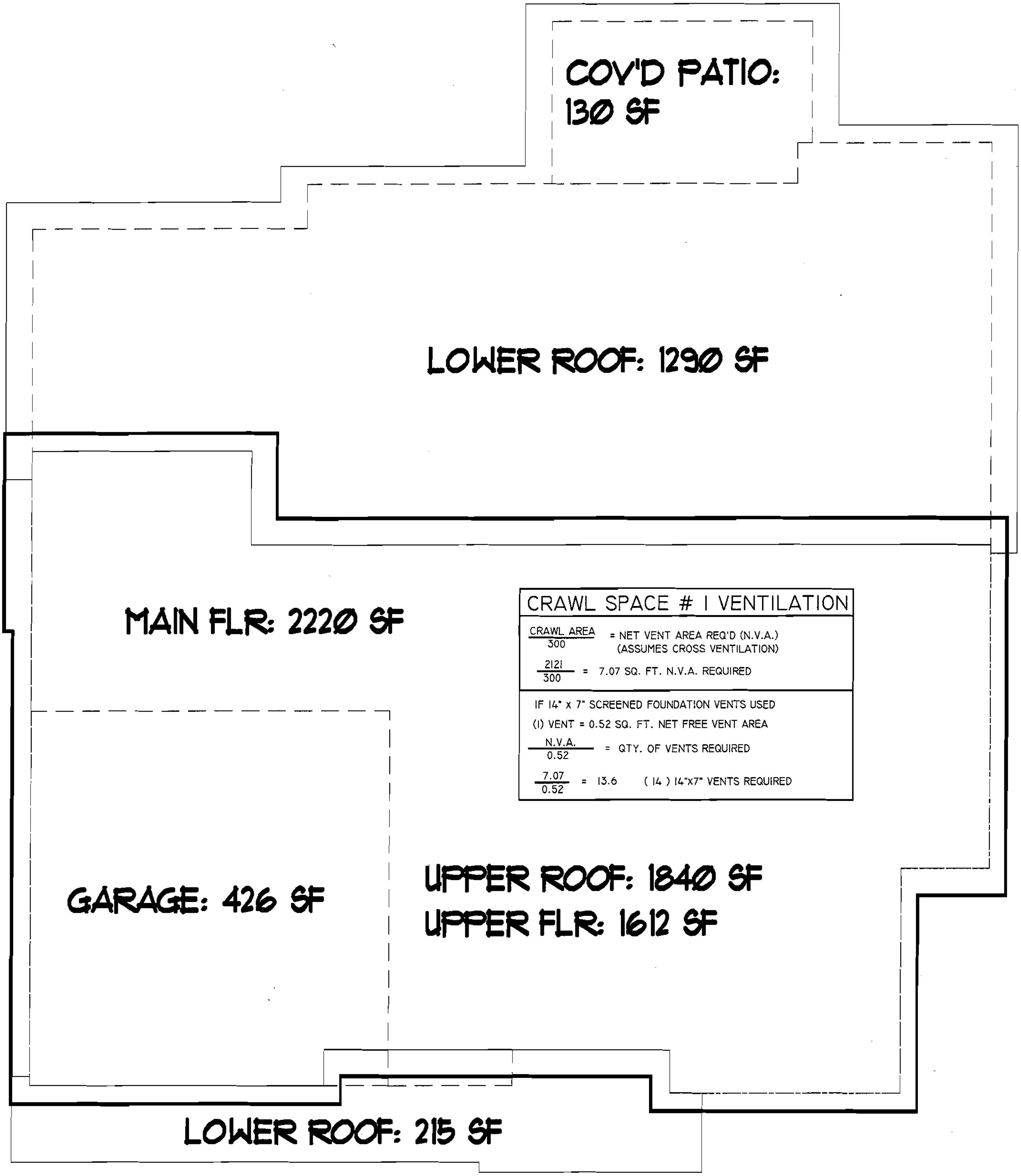
$$w_1 := 15 \cdot psf \cdot A_1 + 12 \cdot psf \cdot 4.5 \cdot ft \cdot P_1$$

Weight of floors include 10psf weight of floor framing, flooring material, insulation, plus 10psf for miscellaneous interior walls.

Story Weight at Main Floor:

$$w_2 := 20 \cdot psf \cdot A_{2a} + 15 psf \cdot A_{2b} + 12 \cdot psf \cdot (4.5 \cdot ft \cdot P_1 + 5 ft \cdot P_2)$$

$$W := w_1 + w_2 = 118996.25\, lb$$


COV'D PATIO: 130 SF
LOWER ROOF: 1290 SF
MAIN FLR: 2220 SF
CRAWL SPACE # 1 VENTILATION
CRAWL AREA / 300 = NET VENT AREA REQ'D (N.V.A.) (ASSUMES CROSS VENTILATION)
2121 / 300 = 7.07 SQ. FT. N.V.A. REQUIRED
IF 14" x 7" SCREENED FOUNDATION VENTS USED
(1) VENT = 0.52 SQ. FT. NET FREE VENT AREA
N.V.A. / 0.52 = QTY. OF VENTS REQUIRED
7.07 / 0.52 = 13.6 (14) 14"x7" VENTS REQUIRED
GARAGE: 426 SF
UPPER ROOF: 1840 SF
UPPER FLR: 1612 SF
LOWER ROOF: 215 SF

Approximate Fundamental Period, T_a:

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

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

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\frac{R}{I_e}\right) \cdot T_a} = 0.41$ (ASCE 7-16 Eq. 12.8-3)

C_s shall not be less than: $0.044 S_{DS} \cdot I_e = 0.04$ (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: $V_E := C_s \cdot W = 16830.34\,\text{lb}$

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

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

$h_1 := 19\text{ft}$ $h_2 := 10\text{ft}$ (Height from base to level x)

$C_{v1} := \dfrac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.49$ $F_1 := C_{v1} \cdot V_E = 8192.17\,\text{lb}$ Story Shear at Upper Floor

$C_{v2} := \dfrac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.51$ $F_2 := C_{v2} \cdot V_E = 8638.17\,\text{lb}$ Story Shear at Main Floor

WIND DESIGN

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

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

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

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

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

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

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

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

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

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

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

Velocity Pressure Exposure Coefficient (Table 26.10-1):

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

$z_1 := 19ft$ $z_2 := 15ft$ Height from ground to level x (z_{min} = 15ft)

$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.89$ $K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.85$ $K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.94$

External Pressure Coefficients w/ Roof Pitch = 5/12 (23 degrees) Front to Back & 5/12 (23 degrees) Side to Side
Taken from Figure 27.3-1

Front to Back:

$L_{fb} := 54ft$ $B_{fb} := 54ft$ $\frac{L_{fb}}{B_{fb}} = 1$ $\frac{h}{L_{fb}} = 0.44$

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

$C_{pf2} := 0.14$ Windward Roof

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

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

Side to Side:

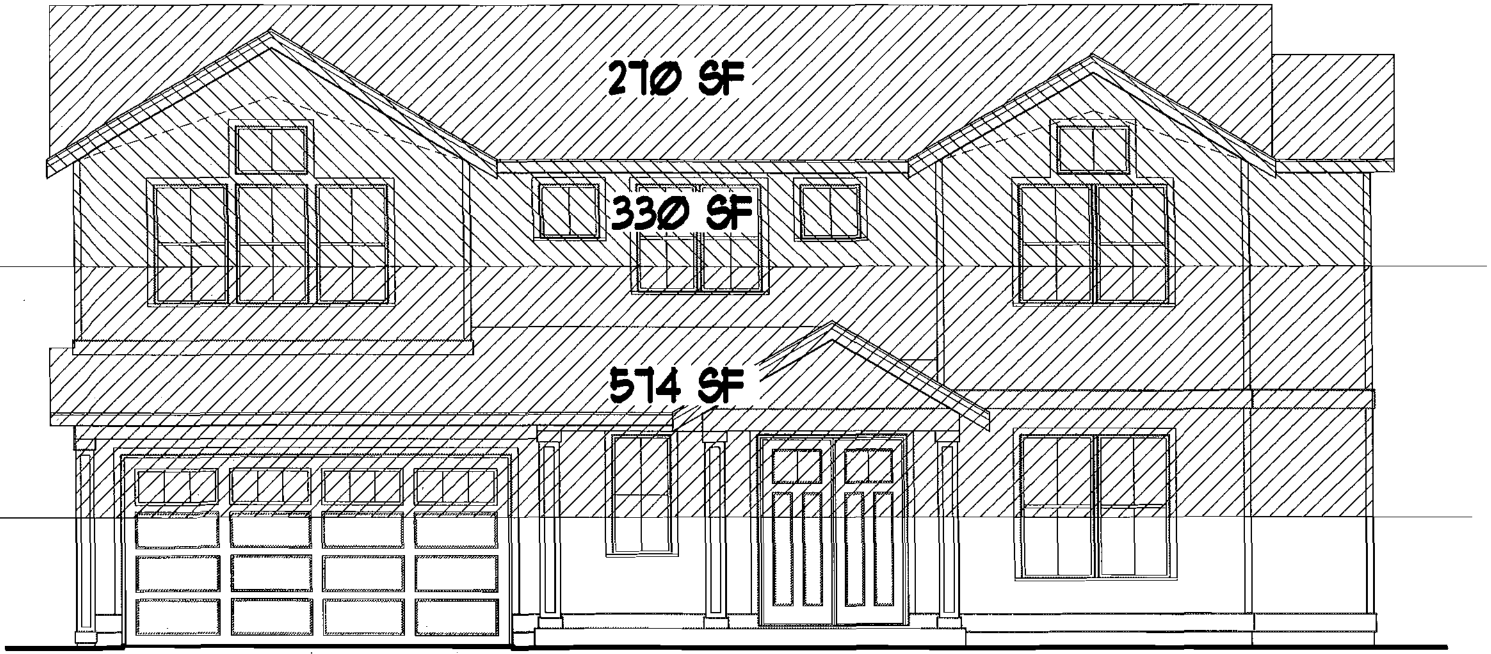
$L_{ss} := 54ft$ $B_{ss} := 54ft$ $\frac{L_{ss}}{B_{ss}} = 1$ $\frac{h}{L_{ss}} = 0.44$

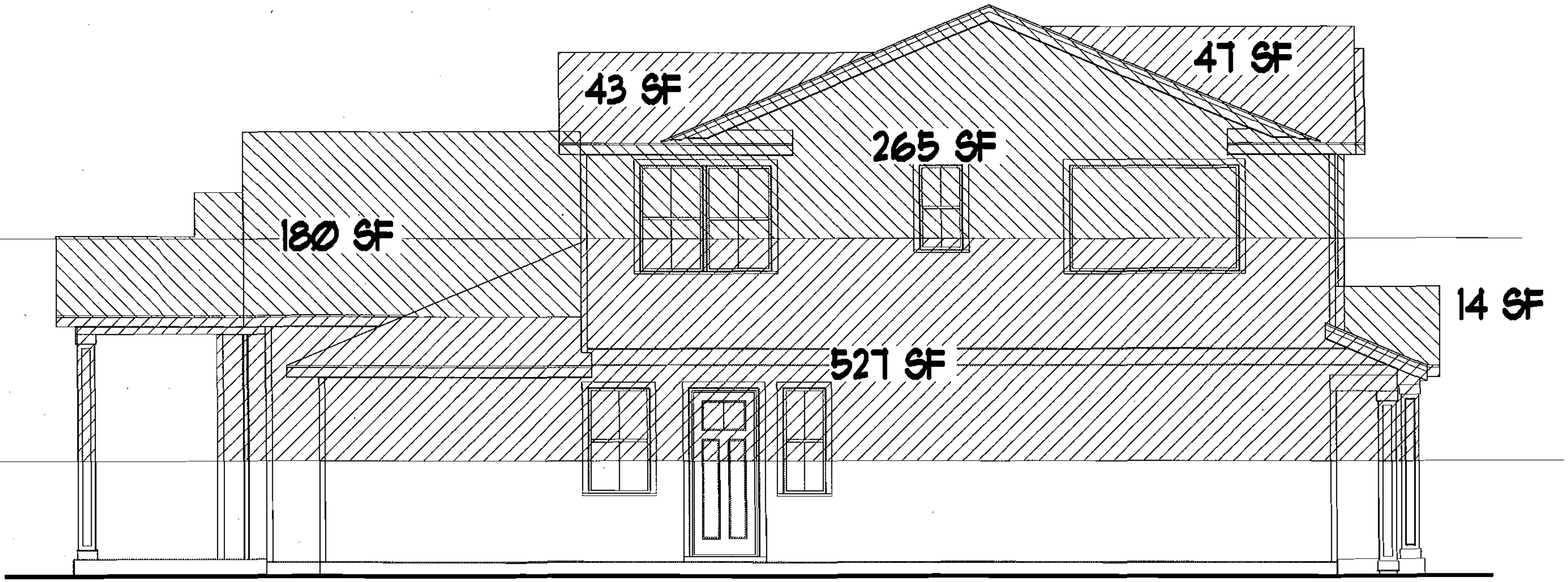
$C_{ps1} := .8$ Windward Wall

$C_{ps2} := 0.14$ Windward Roof

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

$C_{ps4} := -.5$ Leeward Wall


270 SF
330 SF
574 SF


43 SF
47 SF
265 SF
180 SF
14 SF
527 SF

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

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 19.41$ $q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 18.47$ $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e V^2 = 20.39$

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 $p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 13.2\,ft^{-2} \cdot lb$ $p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 12.56\,ft^{-2} \cdot lb$

Windward Roof Front to Back $p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 2.43\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -10.4\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -8.67\,ft^{-2} \cdot lb$

Windward Roof Side to Side $p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 2.43\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -10.4\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -8.67\,ft^{-2} \cdot lb$

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

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

$p_{wr1} - p_{lr1} = 12.83\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw1} = 21.87\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw1} = 21.23\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 12.83\,ft^{-2} \cdot lb$ $p_{ww1} - p_{lw2} = 21.87\,ft^{-2} \cdot lb$ $p_{ww2} - p_{lw2} = 21.23\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})270ft^2 + (p_{ww1} - p_{lw1}) \cdot 330 \cdot ft^2 = 10680\,lb$$

Wind Pressure at Main Floor (Front to Back):

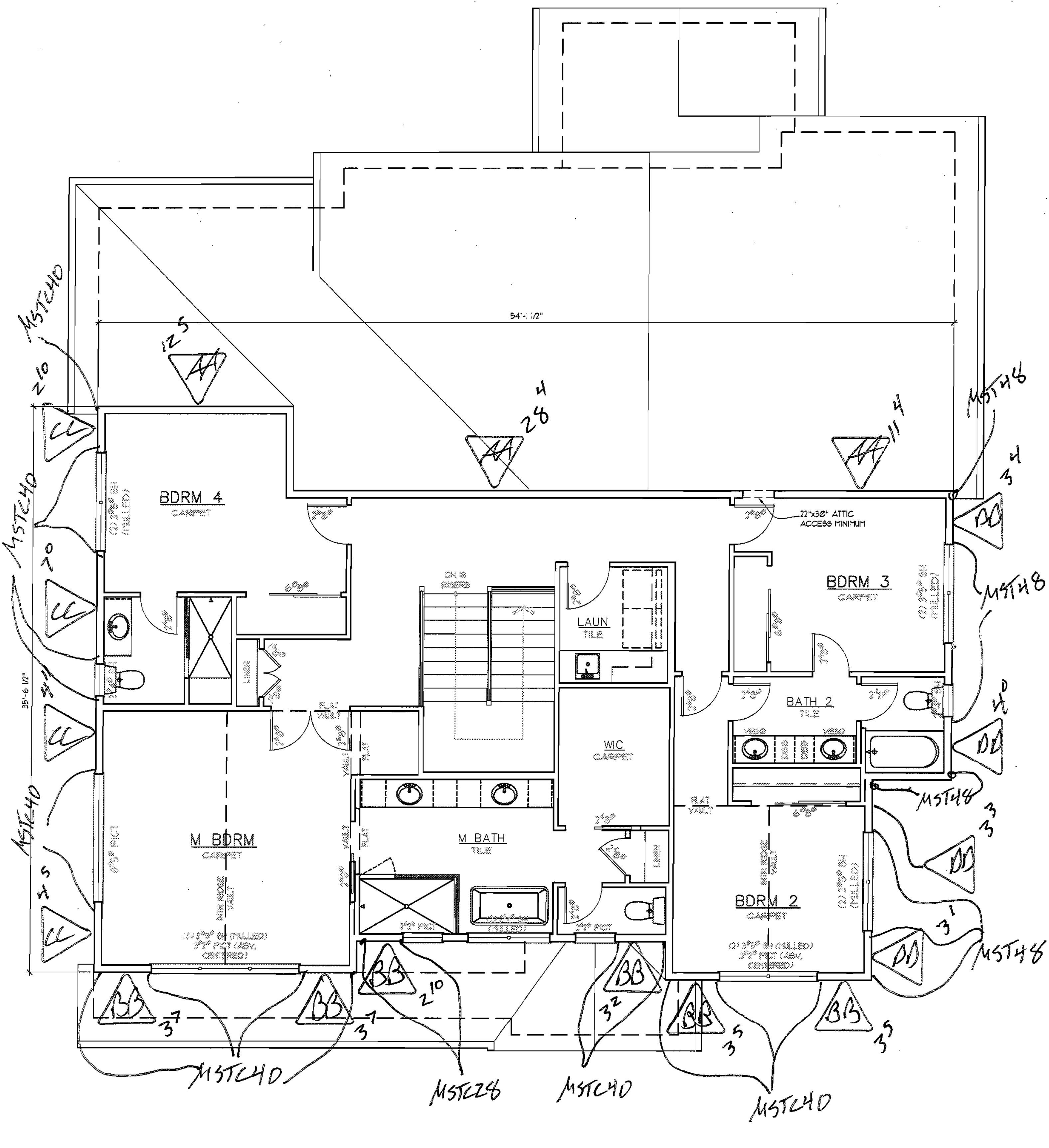
$$V_{2W} := (p_{wr1} - p_{lr1})0ft^2 + (p_{ww2} - p_{lw1}) \cdot 574 \cdot ft^2 = 12184.74\,lb$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 90ft^2 + (p_{ww1} - p_{lw2}) \cdot 265ft^2 = 6949.65\,lb$$

Wind Pressure at Main Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 194ft^2 + (p_{ww2} - p_{lw2}) \cdot 527ft^2 = 13675.5\,lb$$



54'-1 1/2"
35'-6 1/2"
BDRM 4
CARPET
BDRM 3
CARPET
22"x30" ATTIC ACCESS MINIMUM
LAUN
TILE
BATH 2
TILE
WIC
CARPET
M BDRM
CARPET
M BATH
TILE
BDRM 2
CARPET
MSTC40
MSTC28
MST48
AA
BB
CC
DD

WALL AA:

Story Shear due to Wind: $V_{3W} = 6949.65\,lb$ Story Shear due to Seismic: $F_1 = 8192.17\,lb$

Bldg Width in direction of Load: $L_t := 36\cdot ft$ Distance between shear walls: $L_1 := 36\cdot ft$

Shear Wall Length: $Laa_w := (12.42 + 28.33 + 11.33)ft = 52.08\,ft$ $Laa_s := (12.42 + 28.33 + 11.33)ft = 52.08\,ft$

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

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

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

Seismic Force: $\rho := 1.0$ $E_{aa} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Laa_s}$

$vaa = 40.03\,ft^{-1}\cdot lb$ $\frac{vaa}{C_o} = 40.03\,ft^{-1}\cdot lb$

$E_{aa} = 55.05\,ft^{-1}\cdot lb$ $\frac{E_{aa}}{C_o} = 55.05\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{aa} := 11.33\cdot ft$ Plate Height: $Pt := 9\cdot ft$

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

$DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 679.8\,lb$

Chord Force:

$CFaa_w := \frac{vaa\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 360.29\,lb$

$CFaa_s := \frac{E_{aa}\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 495.49\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = -47.59\,lb$

$HDFaa_s := CFaa_s - \left(0.6 - 0.14S_{DS}\right)DLRaa = 175.11\,lb$

No Holdown Required

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

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

$B_p := \frac{(Z_N\cdot C_D\cdot C_o)}{vaa} = 4.08\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{aa}} = 2.96\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vaa} = 34.37\,ft$ $\frac{(Z_B\cdot C_o)}{E_{aa}} = 24.99\,ft$

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

WALL BB:

Story Shear due to Wind: $V_{3W} = 6949.65\,lb$ Story Shear due to Seismic: $F_1 = 8192.17\,lb$

Bldg Width in direction of Load: $L_t := 36\cdot ft$ Distance between shear walls: $L_1 := 36\cdot ft$

Shear Wall Length:

$Lbb_w := (2\cdot 3.58 + 2.83 + 3.17 + 2\cdot 3.42)ft = 20\,ft$ $Lbb_s := \left[2\cdot 3.58\left(\frac{7.17}{9}\right) + 2.83\left(\frac{5.67}{9}\right) + 3.17\left(\frac{6.33}{9}\right) + 2\cdot 3.42\left(\frac{6.83}{9}\right)\right]ft = 14.91\,ft$

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

Wind Force: $vbb := \dfrac{\frac{0.6V_{3W}}{L_t}\cdot\frac{L_1}{2}}{Lbb_w}$ Seismic Force: $\rho := 1.0$ $E_{bb} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Lbb_s}$

$vbb = 104.24\,ft^{-1}\cdot lb$ $\frac{vbb}{C_o} = 104.24\,ft^{-1}\cdot lb$ $E_{bb} = 192.34\,ft^{-1}\cdot lb$ $\frac{E_{bb}}{C_o} = 192.34\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_{bb} := 2.83\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{bb} := (15\cdot psf)\cdot 15\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRbb := \frac{W_{bb}\cdot L_{bb}}{2}$ $DLRbb = 445.72\,lb$

Chord Force:

$CFbb_w := \frac{vbb\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}}$ $CFbb_w = 938.2\,lb$ $CFbb_s := \frac{E_{bb}\cdot L_{bb}\cdot Pt}{C_o\cdot L_{bb}}$ $CFbb_s = 1731.04\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6\cdot DLRbb = 670.77\,lb$ $HDFbb_s := CFbb_s - (0.6 - 0.14S_{DS})\cdot DLRbb = 1520.98\,lb$

Simpson MSTC40 to wall or MSTC28 at flush beam

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vbb} = 1.57\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{bb}} = 0.85\,ft$

16d @ 8" o.c.

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

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

$A_s := \frac{(Z_B\cdot C_o)}{vbb} = 13.2\,ft$ $\frac{(Z_B\cdot C_o)}{E_{bb}} = 7.15\,ft$

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

Myers Engineering, LLC
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PROJECT :**80xx SE 20th Street**

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

WALL CC:

Story Shear due to Wind: $V_{1W} = 10680\,\mathrm{lb}$

Story Shear due to Seismic: $F_1 = 8192.17\,\mathrm{lb}$

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

Distance between shear walls: $L_1 := 54 \cdot \mathrm{ft}$

Shear Wall Length:

$Lcc_w := (4.42 + 4.92 + 7 + 2.83)\mathrm{ft} = 19.17\,\mathrm{ft}$

$$Lcc_s := \left[4.42\left(\frac{8.83}{9}\right) + 4.92 + 7 + 2.83\left(\frac{5.67}{9}\right)\right]\mathrm{ft} = 18.04\,\mathrm{ft}$$

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

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

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

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

$vcc = 167.14\,\mathrm{ft}^{-1} \cdot \mathrm{lb}$ $\quad \dfrac{vcc}{C_o} = 167.14\,\mathrm{ft}^{-1} \cdot \mathrm{lb}$

$E_{cc} = 158.94\,\mathrm{ft}^{-1} \cdot \mathrm{lb}$ $\quad \dfrac{E_{cc}}{C_o} = 158.94\,\mathrm{ft}^{-1} \cdot \mathrm{lb}$

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

Dead Load Resisting Overturning: $L_{cc} := 2.83 \cdot \mathrm{ft}$ Plate Height: $Pt := 9 \cdot \mathrm{ft}$

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

$DLRcc := \dfrac{W_{cc} \cdot L_{cc}}{2}$ $\quad DLRcc = 275.93\,\mathrm{lb}$

Chord Force:

$CFcc_w := \dfrac{vcc \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $\quad CFcc_w = 1504.23\,\mathrm{lb}$

$CFcc_s := \dfrac{E_{cc} \cdot L_{cc} \cdot Pt}{C_o \cdot L_{cc}}$ $\quad CFcc_s = 1430.5\,\mathrm{lb}$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 1338.67\,\mathrm{lb}$

$HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS}) \cdot DLRcc = 1300.46\,\mathrm{lb}$

Simpson MSTC40 to wall or MSTC28 at flush beam

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

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

$B_R := \dfrac{(C_D \cdot Z_N \cdot C_o)}{vcc} = 0.98\,\mathrm{ft}$ $\quad \dfrac{(C_D \cdot Z_N \cdot C_o)}{E_{cc}} = 1.03\,\mathrm{ft}$

16d @ 12" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vcc} = 8.23\,\mathrm{ft}$ $\quad \dfrac{(Z_B \cdot C_o)}{E_{cc}} = 8.66\,\mathrm{ft}$

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

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WALL DD:

Story Shear due to Wind: $V_{1W} = 10680\,lb$

Story Shear due to Seismic: $F_1 = 8192.17\,lb$

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

Distance between shear walls: $L_1 := 54\cdot ft$

Shear Wall Length:

$Ldd_w := (3.33 + 4 + 3.25 + 3.08)ft = 13.66\,ft$

$Ldd_s := \left[3.33\left(\frac{6.67}{9}\right) + 4\left(\frac{8}{9}\right) + 3.25\left(\frac{6.5}{9}\right) + 3.08\left(\frac{6.17}{9}\right)\right]ft = 10.48\,ft$

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

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

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

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

$vdd = 234.55\,ft^{-1}\cdot lb$ $\quad \frac{vdd}{C_o} = 234.55\,ft^{-1}\cdot lb$

$E_{dd} = 273.54\,ft^{-1}\cdot lb$ $\quad \frac{E_{dd}}{C_o} = 273.54\,ft^{-1}\cdot lb$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 495 plf
Seismic Capacity = 353 plf

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

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

$DLRdd := \frac{W_{dd}\cdot L_{dd}}{2}$ $\quad DLRdd = 184.8\,lb$

Chord Force:

$CFdd_w := \frac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $\quad CFdd_w = 2110.98\,lb$

$CFdd_s := \frac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $\quad CFdd_s = 2461.83\,lb$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 2000.1\,lb$

$HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 2374.73\,lb$

Simpson MST48

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

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

$B_P := \frac{(C_D\cdot Z_N\cdot C_o)}{vdd} = 0.7\,ft$ $\quad \frac{(C_D\cdot Z_N\cdot C_o)}{E_{dd}} = 0.6\,ft$

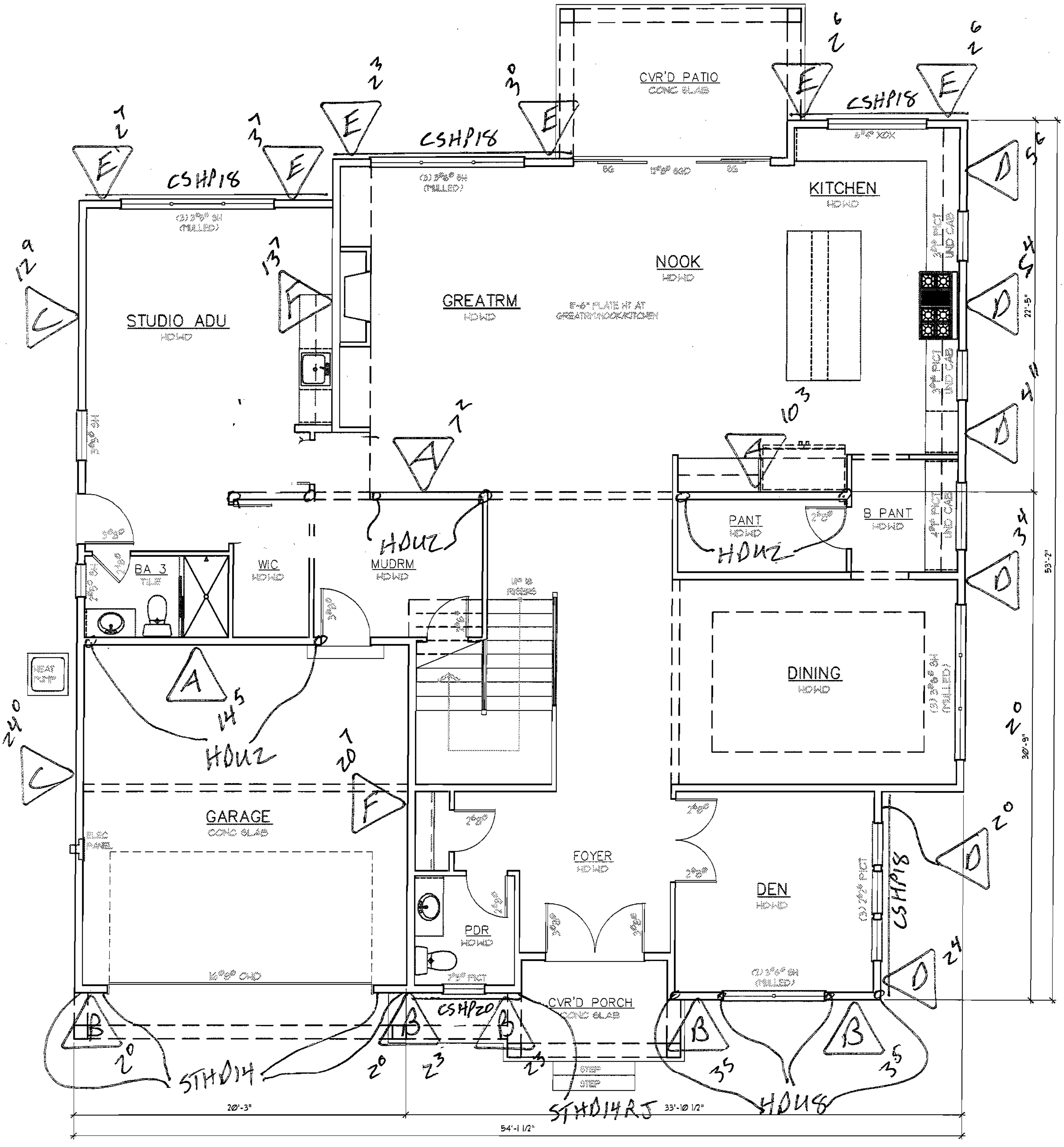
16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vdd} = 5.87\,ft$ $\quad \frac{(Z_B\cdot C_o)}{E_{dd}} = 5.03\,ft$

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


CVR'D PATIO
CONC SLAB
KITCHEN
HDWD
NOOK
HDWD
GREATRM
HDWD
STUDIO ADU
HDWD
BA 3
TILE
WIC
HDWD
MUDRM
HDWD
PANT
HDWD
B PANT
HDWD
DINING
HDWD
GARAGE
CONC SLAB
ELEC PANEL
FOYER
HDWD
DEN
HDWD
PDR
HDWD
CVR'D PORCH
CONC SLAB
STEP
STEP
CSHP18
CSHP18
CSHP18
CSHP18
CSHP20
HDU2
HDU2
HDU2
HDU8
STHD14
STHD14RJ
20'-3"
33'-10 1/2"
54'-1 1/2"
53'-2"
30'-9"
22'-5"

WALL A:

Story Shear due to Wind: $V_{4W} = 13675.5\,lb$ Story Shear due to Seismic: $F_2 = 8638.17\,lb$

Bldg Width in direction of Load: $L_t := 53\cdot ft$ Distance between shear walls: $L_1 := 22.5\cdot ft$ $L_2 := 30.5ft$

Shear Wall Length: $La_w := (14.42 + 7.17 + 10.25)ft = 31.84\,ft$ $La_s := (14.42 + 7.17 + 10.25)ft = 31.84\,ft$

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

Wind Force: $va := \dfrac{vaa\cdot Laa_w + \left(\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{La_w}$ Seismic Force: $\rho := 1.0$ $E_a := \dfrac{E_{aa}\cdot Laa_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{La_s}$

$va = 194.33\,ft^{-1}\cdot lb$ $\frac{va}{C_o} = 194.33\,ft^{-1}\cdot lb$ $E_a = 185.01\,ft^{-1}\cdot lb$ $\frac{E_a}{C_o} = 185.01\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_a := 4.92\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_a := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 5ft$ $DLRa := \frac{W_a\cdot L_a}{2}$ $DLRa = 344.4\,lb$

Chord Force:

$CFa_w := \frac{va\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_w = 1748.99\,lb$ $CFa_w + CFaa_w = 2109.28\,lb$

$CFa_s := \frac{E_a\cdot L_a\cdot Pt}{C_o\cdot L_a}$ $CFa_s = 1665.06\,lb$ $CFa_s + CFaa_s = 2160.56\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6\cdot DLRa = 1542.35\,lb$
$HDFa_w + HDFaa_w = 1494.76\,lb$

$HDFa_s := CFa_s - (0.6 - 0.14S_{DS})\cdot DLRa = 1502.75\,lb$
$HDFa_s + HDFaa_s = 1677.86\,lb$

Simpson LSTHD8RJ or HDU2 w/ SSTB16 or PAB5 anchor

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{va} = 0.84\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_a} = 0.88\,ft$

16d @ 6" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{va} = 7.08\,ft$ $\frac{(Z_B\cdot C_o)}{E_a} = 7.44\,ft$

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

WALL B:

Story Shear due to Wind: $V_{4W} = 13675.5\,\text{lb}$ Story Shear due to Seismic: $F_2 = 8638.17\,\text{lb}$

Bldg Width in direction of Load: $L_t := 53\cdot\text{ft}$ Distance between shear walls: $L_1 := 31\cdot\text{ft}$

Shear Wall Length: $Lb_w := (2\cdot2 + 2\cdot2.25 + 2\cdot3.42)\text{ft} = 15.34\,\text{ft}$ $Lb_s := \left[2\cdot2 + 2\cdot2.25 + 2\cdot3.42\left(\frac{6.83}{9}\right)\right]\text{ft} = 13.69\,\text{ft}$

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

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

$vb = 292.34\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{vb}{C_o} = 292.34\,\text{ft}^{-1}\cdot\text{lb}$ $E_b = 338.6\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_b}{C_o} = 338.6\,\text{ft}^{-1}\cdot\text{lb}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 495 plf
Seismic Capacity = 353 plf

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

Restraint Panel Height = 10ft Maximum

Restraint Panel Width = 2ft-0in Minimum

Allowable Shear per Panel = 1046 lbs Seismic & 1465 lbs Wind

Shear per Panel: $V_{s1} := (2\text{ft}\cdot E_b) = 677.19\,\text{lb}$ O.K.

$V_{s2} := (2\text{ft}\cdot vb) = 584.69\,\text{lb}$ O.K.

Dead Load Resisting Overturning: $L_b := 3.42\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_b := (15\cdot\text{psf})\cdot0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot7\text{ft}$ $DLRb := \frac{W_b\cdot L_b}{2}$ $DLRb = 290.7\,\text{lb}$

Chord Force:

$CFb_w := \frac{vb\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_w = 2923.44\,\text{lb}$ $CFb_w + CFbb_w = 3861.64\,\text{lb}$

$CFb_s := \frac{E_b\cdot L_b\cdot Pt}{C_o\cdot L_b}$ $CFb_s = 3385.95\,\text{lb}$ $CFb_s + CFbb_s = 5117\,\text{lb}$

Holdown Force:

$HDFb_w := CFb_w - 0.6\cdot DLRb = 2749.02\,\text{lb}$ $HDFb_s := CFb_s - (0.6 - 0.14S_{DS})\cdot DLRb = 3248.95\,\text{lb}$

Simpson STHD14/RJ

$HDFb_w + HDFbb_w = 3419.79\,\text{lb}$ $HDFb_s + HDFbb_s = 4769.92\,\text{lb}$

Simpson HDU8 w/ SB7/8x24 Anchor

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vb} = 0.56\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_b} = 0.48\,\text{ft}$

16d @ 6" o.c.

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

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

$A_s := \frac{(Z_B\cdot C_o)}{vb} = 4.71\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_b} = 4.06\,\text{ft}$

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

WALL C:

Story Shear due to Wind: $V_{2W} = 12184.74\,lb$

Story Shear due to Seismic: $F_2 = 8638.17\,lb$

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

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

Shear Wall Length: $Lc_w := (12.75 + 24)ft = 36.75\,ft$ $\quad Lc_s := (12.75 + 24)ft = 36.75\,ft$

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

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

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

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

$vc = 124.02\,ft^{-1} \cdot lb$ $\quad \dfrac{vc}{C_o} = 124.02\,ft^{-1} \cdot lb$

$E_c = 108.49\,ft^{-1} \cdot lb$ $\quad \dfrac{E_c}{C_o} = 108.49\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_c := 12.75 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_c := (15 \cdot psf) \cdot 5 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$

$DLRc := \dfrac{W_c \cdot L_c}{2}$ $\quad DLRc = 1051.87\,lb$

Chord Force:

$CFc_w := \dfrac{vc \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $\quad CFc_w = 1116.21\,lb$

$CFc_w + CFcc_w = 2620.44\,lb$

$CFc_s := \dfrac{E_c \cdot L_c \cdot Pt}{C_o \cdot L_c}$ $\quad CFc_s = 976.41\,lb$

$CFc_s + CFcc_s = 2406.91\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6 \cdot DLRc = 485.09\,lb$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS}) \cdot DLRc = 480.67\,lb$

No Holdown Required

$HDFc_w + HDFcc_w = 1823.76\,lb$

$HDFc_s + HDFcc_s = 1781.13\,lb$

Simpson LSTHD8/RJ

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

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

$B_p := \dfrac{(C_D \cdot Z_N \cdot C_o)}{vc} = 1.32\,ft$ $\quad \dfrac{(C_D \cdot Z_N \cdot C_o)}{E_c} = 1.5\,ft$

16d @ 16" o.c.

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

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

$As := \dfrac{(Z_B \cdot C_o)}{vc} = 11.09\,ft$ $\quad \dfrac{(Z_B \cdot C_o)}{E_c} = 12.68\,ft$

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

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

WALL D:

Story Shear due to Wind: $V_{2W} = 12184.74\,\text{lb}$ Story Shear due to Seismic: $F_2 = 8638.17\,\text{lb}$

Bldg Width in direction of Load: $L_t := 54\cdot\text{ft}$ Distance between shear walls: $L_1 := 34\cdot\text{ft}$

Shear Wall Length: $Ld_w := (5.5 + 5.33 + 4.92 + 3.33 + 2 + 2.33)\text{ft} = 23.41\,\text{ft}$

$$Ld_s := \left[5.5\left(\frac{11}{11.5}\right) + 5.33\left(\frac{10.67}{11.5}\right) + 4.92\left(\frac{9.83}{11.5}\right) + 3.33\left(\frac{6.67}{9}\right) + 2 + 2.33\right]\text{ft} = 21.21\,\text{ft}$$

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

Wind Force: $vd := \dfrac{vdd\cdot Ldd_w + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Ld_w}$ Seismic Force: $\rho := 1.0$ $E_d := \dfrac{E_{dd}\cdot Ldd_s + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Ld_s}$

$vd = 235.18\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{vd}{C_o} = 235.18\,\text{ft}^{-1}\cdot\text{lb}$ $E_d = 224.94\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_d}{C_o} = 224.94\,\text{ft}^{-1}\cdot\text{lb}$

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

Dead Load Resisting Overturning: $L_d := 3.33\cdot\text{ft}$ Plate Height: $Pt := 9\cdot\text{ft}$

$W_d := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 1\text{ft}$ $DLRd := \frac{W_d\cdot L_d}{2}$ $DLRd = 166.5\,\text{lb}$

Chord Force:

$CFd_w := \frac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_w = 2116.62\,\text{lb}$ $CFd_s := \frac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_s = 2024.45\,\text{lb}$

$CFd_w + CFdd_w = 4227.6\,\text{lb}$ $CFd_s + CFdd_s = 4486.27\,\text{lb}$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 2016.72\,\text{lb}$ $HDFd_s := CFd_s - (0.6 - 0.14S_{DS})\cdot DLRd = 1945.98\,\text{lb}$

Simpson LSTHD8RJ

$HDFd_w + HDFdd_w = 4016.82\,\text{lb}$ $HDFd_s + HDFdd_s = 4320.71\,\text{lb}$

Simpson HDU5 w/ SB5/8x24 anchor

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

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

$B_R := \frac{(C_D\cdot Z_N\cdot C_o)}{vd} = 0.69\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_d} = 0.73\,\text{ft}$

16d @ 8" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{vd} = 5.85\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_d} = 6.12\,\text{ft}$

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

WALL E:

Story Shear due to Wind: $V_{4W} = 13675.5\,lb$ Story Shear due to Seismic: $F_2 = 8638.17\,lb$

Bldg Width in direction of Load: $L_t := 54\cdot ft$ Distance between shear walls: $L_1 := 23\cdot ft$

Shear Wall Length:

$Le_w := (2.58 + 3.58 + 2.25 + 3 + 2\cdot 2.5)ft = 16.41\,ft$

$Le_s := \left[2.58 + 3.58 + 2.25\left(\frac{4.5}{8}\right) + 3\left(\frac{6}{8}\right) + 2\cdot 2.5\left(\frac{5}{6.5}\right)\right]ft = 13.52\,ft$

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

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

Wind Force: $ve := \dfrac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1}{2}}{Le_w}$ Seismic Force: $\rho := 1.0$ $E_e := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}}{Le_s}$

$ve = 106.49\,ft^{-1}\cdot lb$ $\frac{ve}{C_o} = 106.49\,ft^{-1}\cdot lb$ $E_e = 95.23\,ft^{-1}\cdot lb$ $\frac{E_e}{C_o} = 95.23\,ft^{-1}\cdot lb$

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

Dead Load Resisting Overturning: $L_e := 11\cdot ft$ Plate Height: $Pt := 11.5\cdot ft$

$W_e := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 797.5\,lb$

Chord Force:

$CFe_w := \frac{ve\cdot 5ft\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 556.63\,lb$ $CFe_s := \frac{E_e\cdot 5ft\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 497.81\,lb$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 78.13\,lb$ $HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 121.96\,lb$

No Holdown Required

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

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

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 1.53\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 1.71\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B\cdot C_o)}{ve} = 12.92\,ft$ $\frac{(Z_B\cdot C_o)}{E_e} = 14.45\,ft$

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

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WALL F:

Story Shear due to Wind: $V_{2W} = 12184.74\,lb$ Story Shear due to Seismic: $F_2 = 8638.17\,lb$

Bldg Width in direction of Load: $L_t := 54 \cdot ft$ Distance between shear walls: $L_1 := 20 \cdot ft$ $L_2 := 34ft$

Shear Wall Length: $Lf_w := (13.58 + 20.58)ft = 34.16\,ft$ $Lf_s := (13.58 + 20.58)ft = 34.16\,ft$

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

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

Wind Force: $vf := \frac{\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf_w}$

Seismic Force: $\rho := 1.0$ $E_f := \frac{\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}}{Lf_s}$

$vf = 107.01\,ft^{-1} \cdot lb$ $\frac{vf}{C_o} = 107.01\,ft^{-1} \cdot lb$ $E_f = 88.51\,ft^{-1} \cdot lb$ $\frac{E_f}{C_o} = 88.51\,ft^{-1} \cdot lb$

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

Dead Load Resisting Overturning: $L_f := 13.58 \cdot ft$ Plate Height: $Pt := 11.5 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 848.75\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 1230.6\,lb$ $CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 1017.82\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 721.35\,lb$ $HDFf_s := CFf_s - (0.6 - 0.14S_{DS}) \cdot DLRf = 617.81\,lb$

No Holdown Required

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

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

$B_p := \frac{(C_D \cdot Z_N \cdot C_o)}{vf} = 1.53\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_f} = 1.84\,ft$

16d @ 16" o.c.

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

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

$As := \frac{(Z_B \cdot C_o)}{vf} = 12.86\,ft$ $\frac{(Z_B \cdot C_o)}{E_f} = 15.55\,ft$

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

Diapragm Shear Check:

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

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

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

Wall Lines AA:

$$vaa\cdot\frac{Laa_w}{54ft} = 38.61\,ft^{-1}\cdot lb \qquad E_{aa}\cdot\frac{Laa_s}{54ft} = 53.1\,ft^{-1}\cdot lb$$

Wall Lines BB:

$$vbb\cdot\frac{Lbb_w}{54ft} = 38.61\,ft^{-1}\cdot lb \qquad E_{bb}\cdot\frac{Lbb_s}{54ft} = 53.1\,ft^{-1}\cdot lb$$

Wall Lines CC:

$$vcc\cdot\frac{Lcc_w}{3556ft} = 0.9\,ft^{-1}\cdot lb \qquad E_{cc}\cdot\frac{Lcc_s}{35ft} = 81.92\,ft^{-1}\cdot lb$$

Wall Lines DD:

$$vdd\cdot\frac{Ldd_w}{31ft} = 103.35\,ft^{-1}\cdot lb \qquad E_{dd}\cdot\frac{Ldd_s}{31ft} = 92.49\,ft^{-1}\cdot lb$$

Wall Lines A:

$$\frac{va\cdot La_w - vaa\cdot Laa_w}{54ft} = 75.98\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La_s - E_{aa}\cdot Laa_s}{54ft} = 55.99\,ft^{-1}\cdot lb \qquad \frac{va\cdot La_w}{54ft} = 114.58\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La_s}{54ft} = 109.09\,ft^{-1}\cdot lb$$

Wall Lines B:

$$\frac{vb\cdot Lb_w}{54ft} = 83.05\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb_s}{54ft} = 85.85\,ft^{-1}\cdot lb$$

Wall Lines C:

$$\frac{vc\cdot Lc_w - vcc\cdot Lcc_w}{48ft} = 28.21\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc_s - E_{cc}\cdot Lcc_s}{48ft} = 23.33\,ft^{-1}\cdot lb \qquad \frac{vc\cdot Lc_w}{48ft} = 94.96\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc_s}{48ft} = 83.06\,ft^{-1}\cdot lb$$

Wall Lines D:

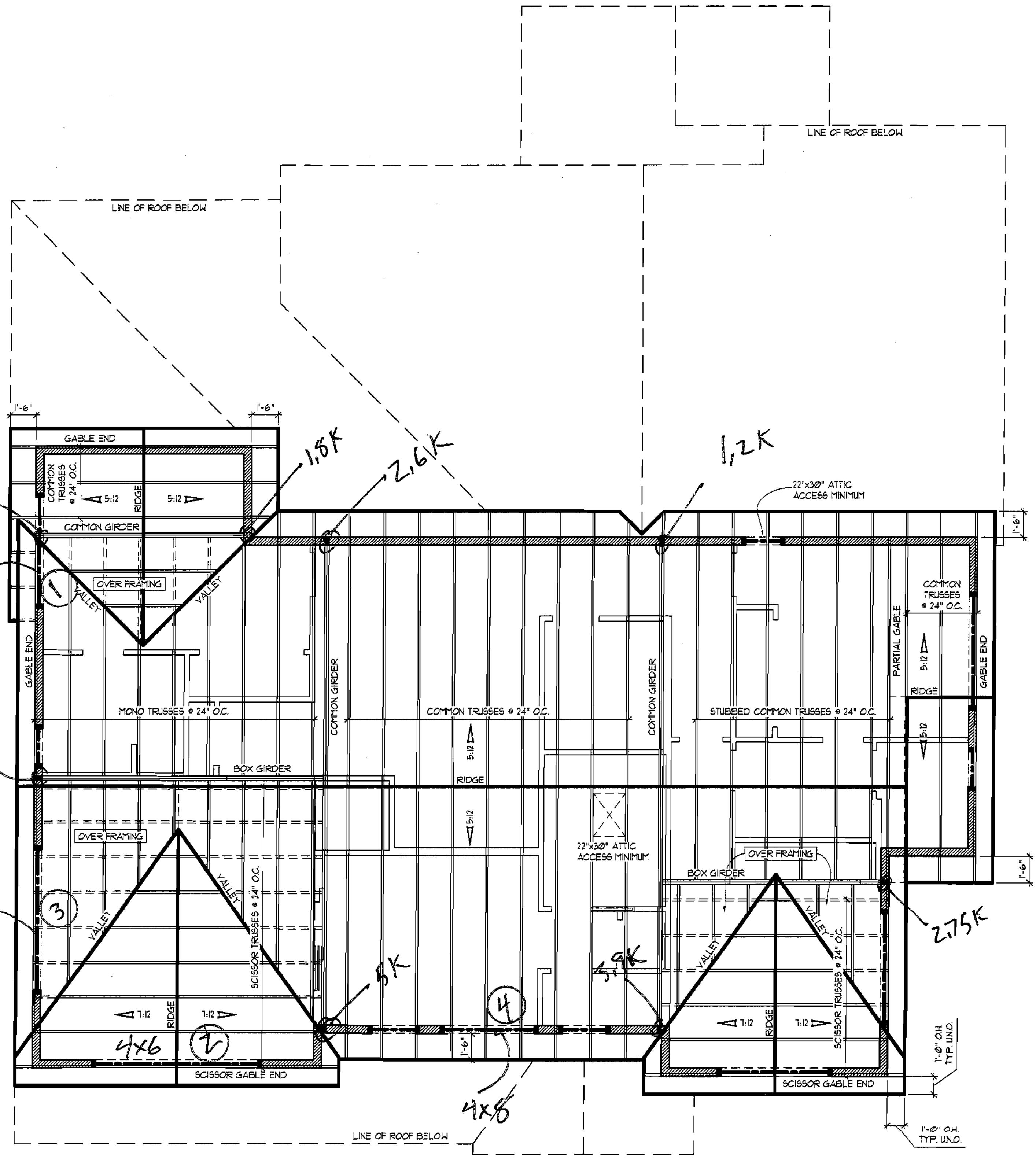
$$\frac{vd\cdot Ld_w - vdd\cdot Ldd_w}{53ft} = 43.43\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld_s - E_{dd}\cdot Ldd_s}{53ft} = 35.92\,ft^{-1}\cdot lb \qquad \frac{vd\cdot Ld_w}{53ft} = 103.88\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld_s}{53ft} = 90.02\,ft^{-1}\cdot lb$$

Wall Line E:

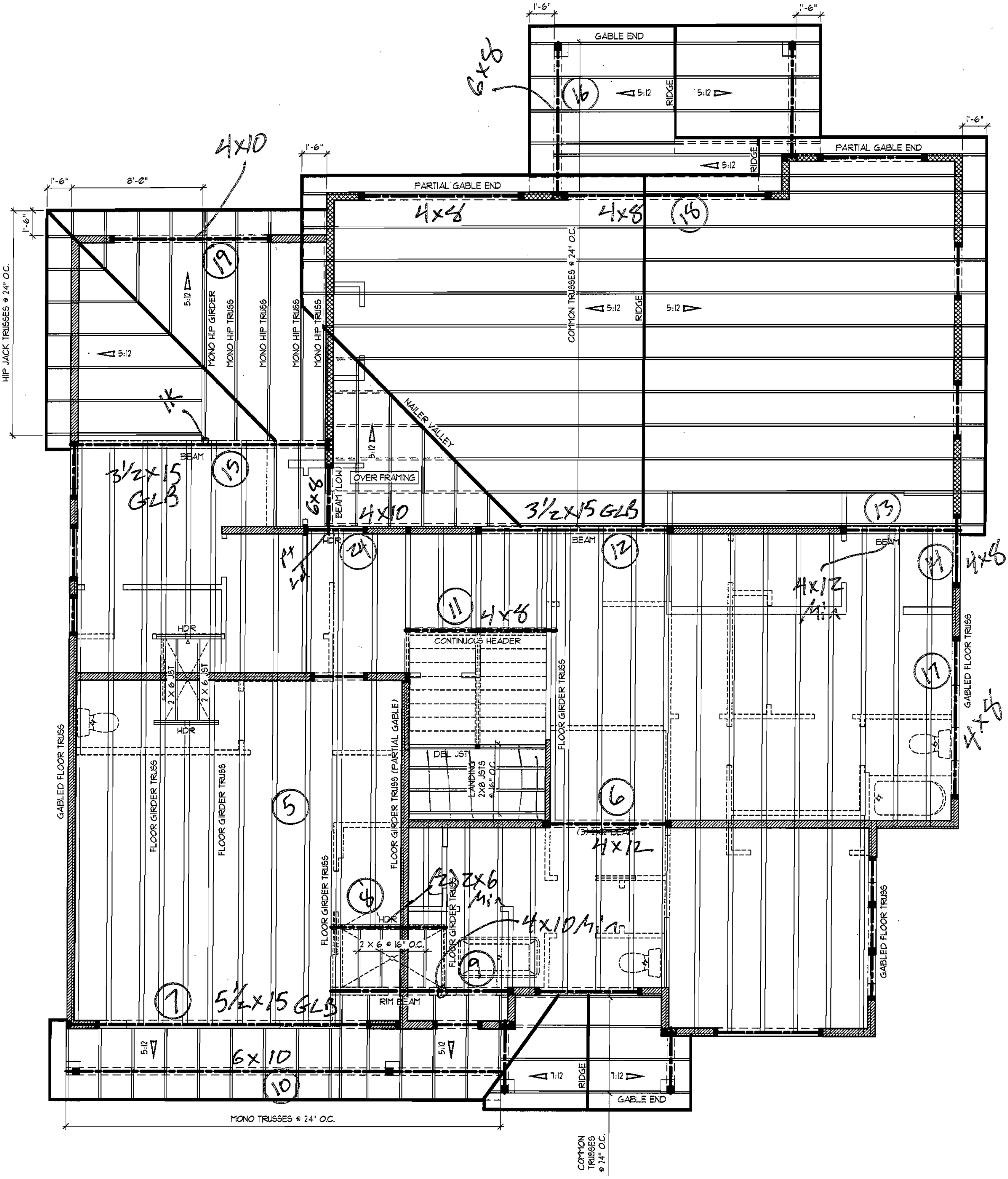
$$\frac{ve\cdot Le_w}{54ft} = 32.36\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le_s}{54ft} = 23.85\,ft^{-1}\cdot lb$$

Wall Line F:

$$\frac{vf\cdot Lf_w}{50ft} = 73.11\,ft^{-1}\cdot lb \qquad \frac{E_f\cdot Lf_s}{50ft} = 60.47\,ft^{-1}\cdot lb$$



LINE OF ROOF BELOW
LINE OF ROOF BELOW
1'-6"
1'-6"
GABLE END
COMMON TRUSSES @ 24" O.C.
5:12
RIDGE
5:12
COMMON GIRDER
1.8K
1.8K
2.6K
1.2K
22"x30" ATTIC ACCESS MINIMUM
4x8
OVER FRAMING
VALLEY
VALLEY
GABLE END
MONO TRUSSES @ 24" O.C.
COMMON GIRDER
COMMON TRUSSES @ 24" O.C.
5:12
COMMON GIRDER
STUBBED COMMON TRUSSES @ 24" O.C.
PARTIAL GABLE
COMMON TRUSSES @ 24" O.C.
5:12
RIDGE
GABLE END
1'-6"
2.5K
BOX GIRDER
RIDGE
5:12
OVER FRAMING
22"x30" ATTIC ACCESS MINIMUM
OVER FRAMING
BOX GIRDER
4x8
VALLEY
VALLEY
SCISSOR TRUSSES @ 24" O.C.
2.75K
VALLEY
VALLEY
SCISSOR TRUSSES @ 24" O.C.
5K
3.9K
7:12
RIDGE
7:12
7:12
RIDGE
7:12
4x6
1'-6"
SCISSOR GABLE END
SCISSOR GABLE END
1'-0" O.H. TYP. U.N.O.
4x8
LINE OF ROOF BELOW
1'-0" O.H. TYP. U.N.O.



DROPPED FRAMING FOR FLUSH ENTRY SHOWERS: PROVIDE 2X6 LEDGERS & BLOCKING AROUND PERIMETER TO ACCEPT EDGE NAILING. SECURE 2X6 TO PERIMETER FRAMING W/ 10d COMMON NAILS (0.148"x3") STAGGERED AT 6" O.C.

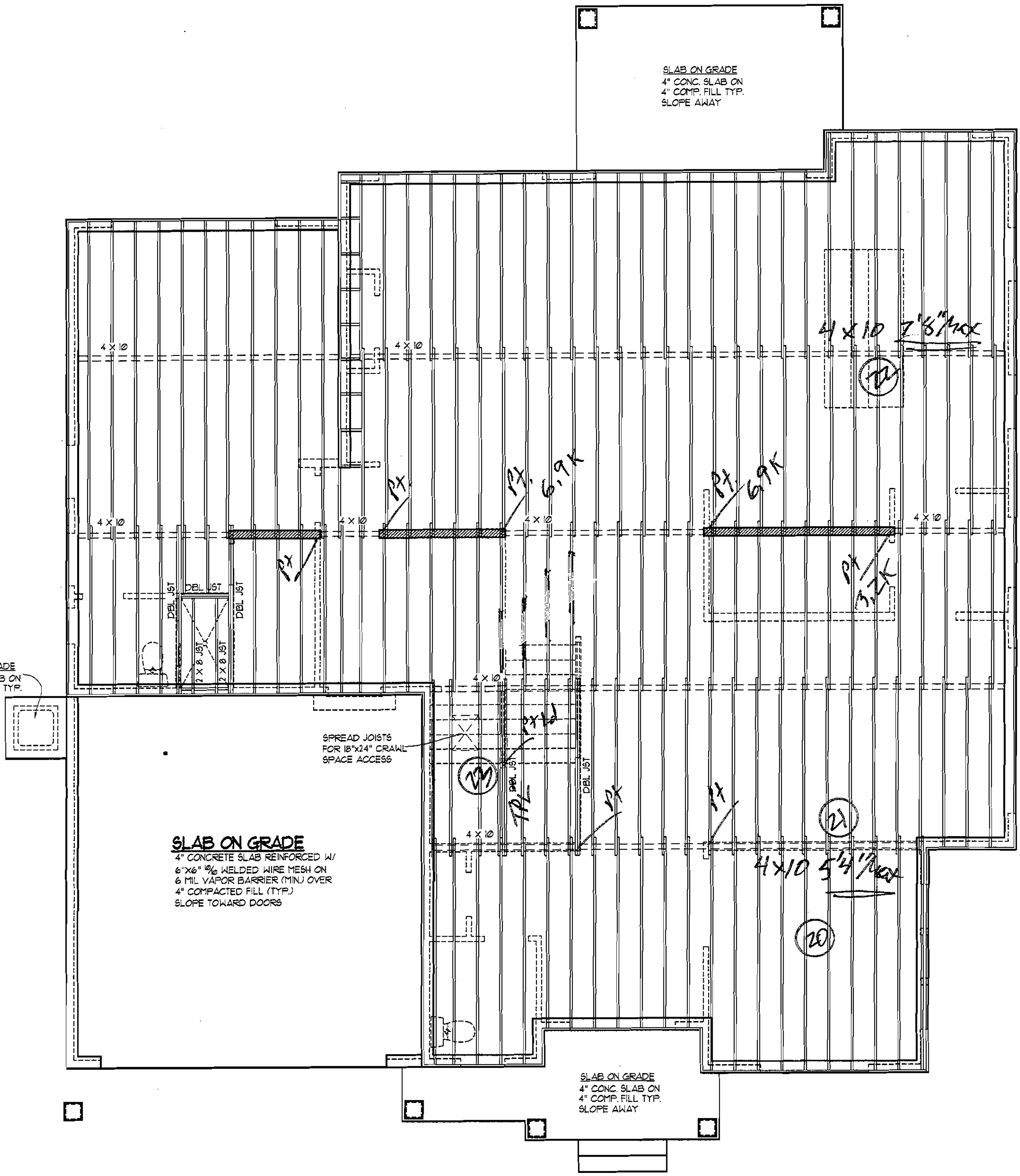
5

15" Min

UPPER FLOOR JOISTS SHALL BE: ~~16.5~~" FLOOR TRUSSES @ 24" O.C. UNLESS NOTED OTHERWISE (U.N.O.)

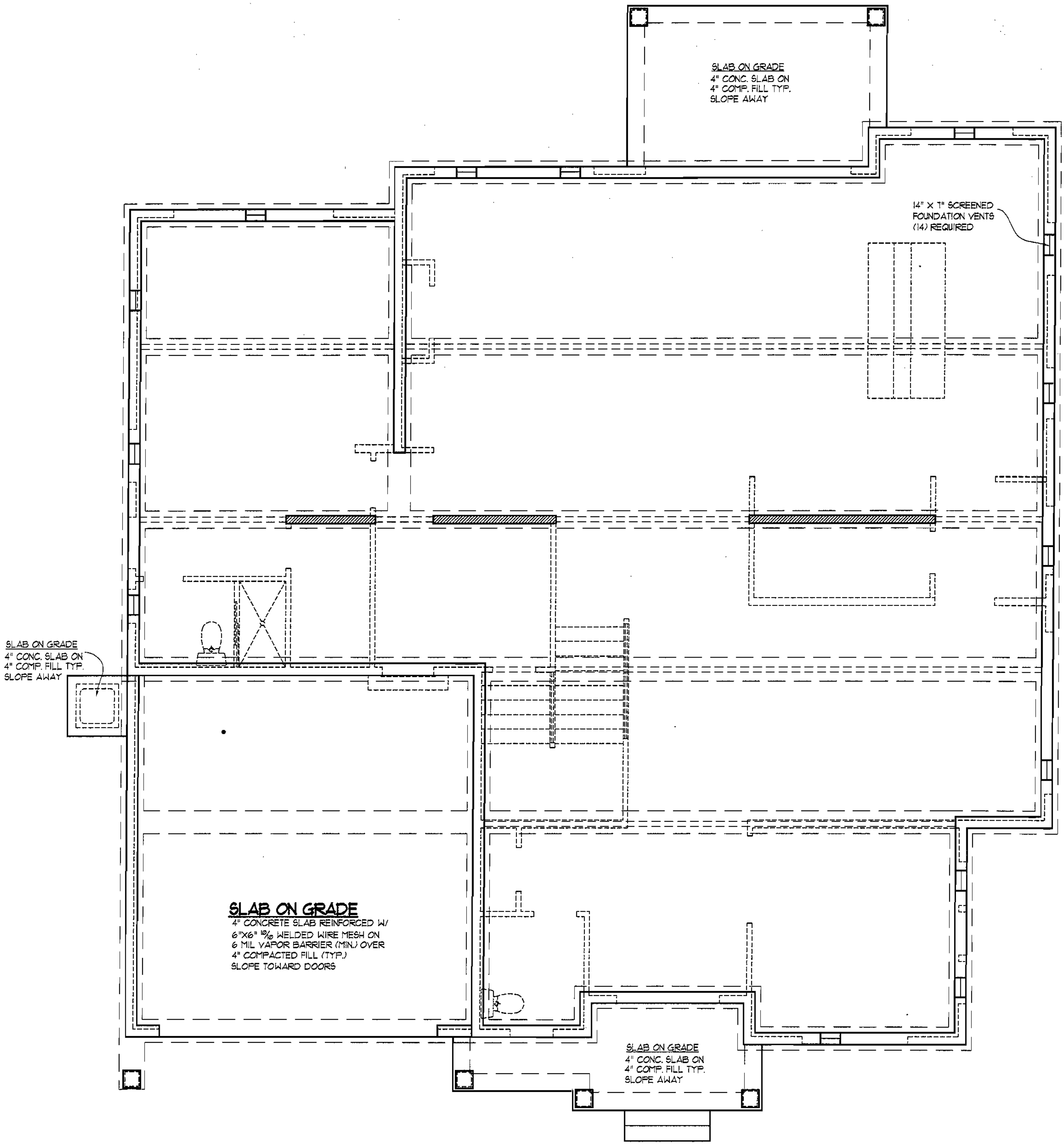
9'-1 1/8" PLATE HT. BEARING WALL

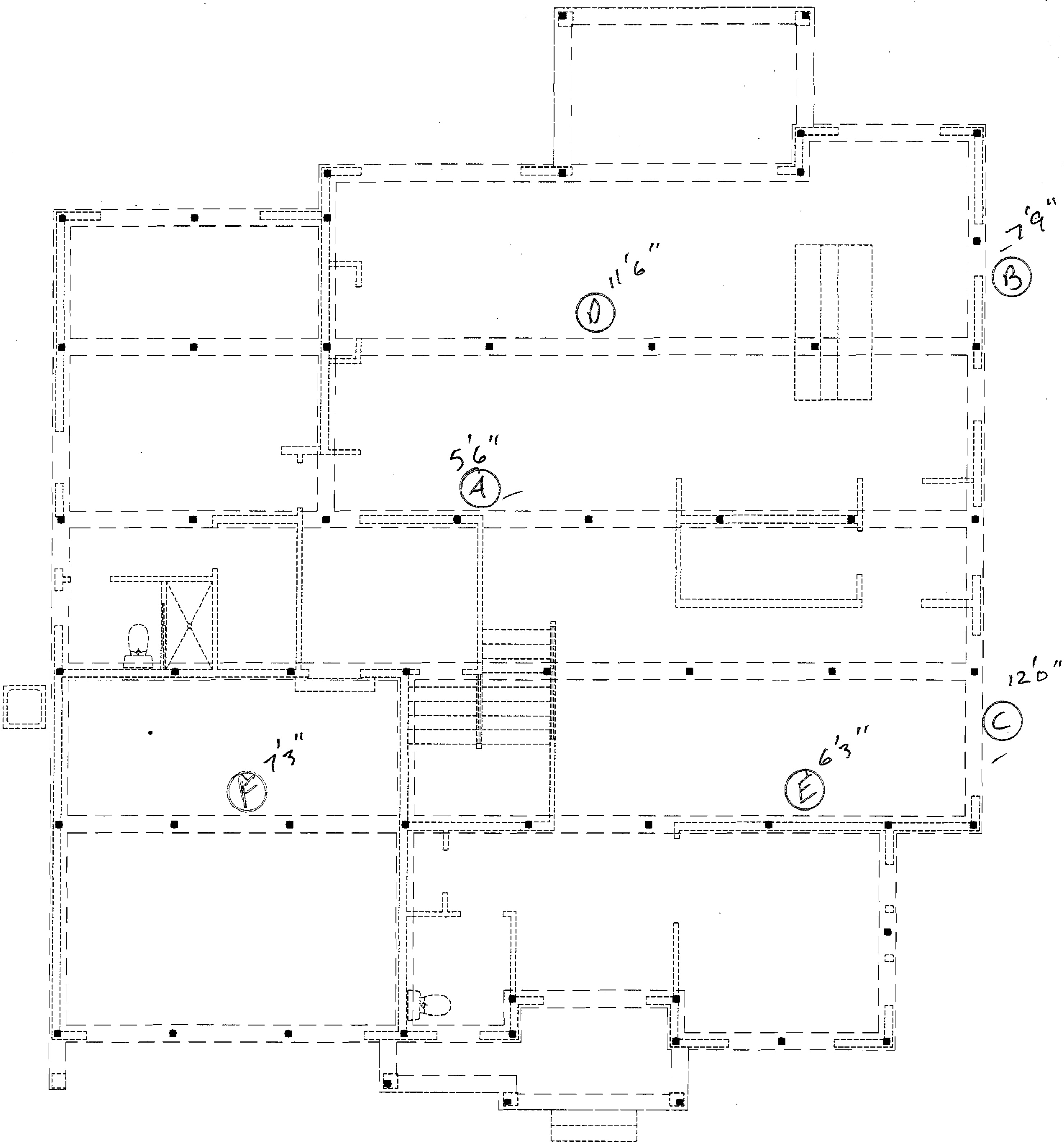
11'-6" PLATE HT. BEARING WALL



20 MAIN FLOOR JOISTS SHALL BE:
2 X 10 HF # 2 JOISTS @ 16" O.C.
UNLESS NOTED OTHERWISE (U.N.O.)

DROPPED FRAMING FOR FLUSH ENTRY SHOWERS:
PROVIDE 2X6 LEDGERS & BLOCKING AROUND
PERIMETER TO ACCEPT EDGE NAILING. SECURE
2X6 TO PERIMETER FRAMING W/ 10d COMMON
NAILS (0.148"x3") STAGGERED AT 6" O.C.


SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
14" X 7" SCREENED
FOUNDATION VENTS
(14) REQUIRED
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY
SLAB ON GRADE
4" CONCRETE SLAB REINFORCED W/
6"X6" 10/10 WELDED WIRE MESH ON
6 MIL VAPOR BARRIER (MIN.) OVER
4" COMPACTED FILL (TYP.)
SLOPE TOWARD DOORS
SLAB ON GRADE
4" CONC. SLAB ON
4" COMP. FILL TYP.
SLOPE AWAY



7'9"
B
11'6"
D
5'6"
A
12'0"
C
7'3"
F
6'3"
E

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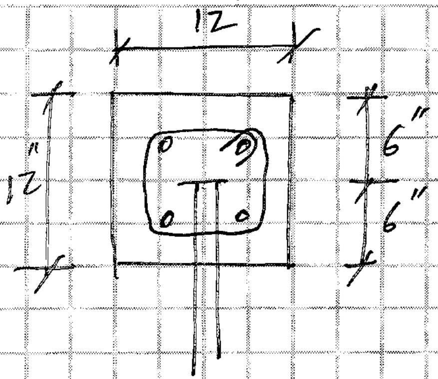
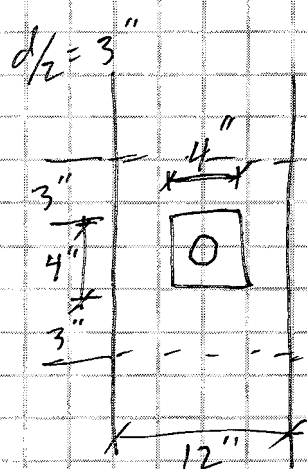
Assume
12"x12" Grade Beam



$b_w = 12"$ $d = 6"$

$f'_c = 2500\,psi$ $\sqrt{f'_c} = 50$



$\beta = 1$ $\lambda = 1$ $V_c = 4\lambda\sqrt{f'_c}\, b_o d$ (11-33)

$b_o = 20$ $d = 6$ $V_c = 4(1)(50)(20)(6) = 24{,}000^{lb}$

$V_u = 20{,}000^{lb}$ for 4" dia. pile

$V_u = 12{,}000^{lb}$ for 3" dia pile

$0.75(24000) = 18{,}000 > 12{,}000$ ∴ OK

Final Grade Beam 12"W x 16"H

$d = 10"$ $V_c = 4(1)(50)(24)(10) = 48{,}000^{lb}$
$b_o = 24"$

FOR Pile Punch @ Grade beam DATE 3-30-21
JOB BY MM

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Grade Beams

(A)
$w_{D_1} = 240\,plf$ (Roof)
$w_{S_1} = 400\,plf$

$w_{D2} = 108\,plf$ (Ext. Wall)

$w_{D3} = 135\,plf$ (upper Flr)
$w_{L3} = 360\,plf$

$w_{D4} = 143\,plf$ (Main Flr)
$w_{L4} = 380\,plf$

$w_{D_{Total}} = 626\,plf + self$
$w_{L_{Total}} = 740\,plf$
$w_{S_{Total}} = 400\,plf$

5'6" span Max

(B)
$w_D = 15psf(42'/2) + 12psf(11.5') + 15psf(1') = 468\,plf + self$
$w_L = 40\,plf$
$w_S = 25psf(42'/2) = 525\,plf$

7'9" span Max

(C)
$w_D = 15psf(2'+1'+1') + 12psf(9'+9') = 276\,plf + self$
$w_L = 40psf(1'+1') = 80\,plf$
$w_S = 25psf(2') = 50\,plf$

12'0" span Max

(D)
$w_D = 165\,plf + self$
$w_L = 440\,plf$

11'6" Max span

(E)
$w_D = 382.5\,plf$
$w_L = 870\,plf$

6'3" span Max

FOR 80xx SE 20th
JOB

DATE 3-31-21
BY MM

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F $w_D = 50psf(20/2) = 500plf + self$ 7'3" span OK

$w_L = 50psf(20/2) = 500plf$

FOR 80xx SE 20th DATE 3-31-21

JOB BY MM

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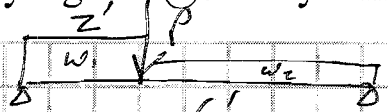
① $w_{D_1} = 15psf(16'/2) = 120plf$
$w_{S_1} = 25psf(16'/2) = 200plf$

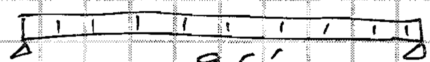
$P = 730^{\#}DL + 1070^{\#}SL$ from Girder

$w_{D_2} = 15psf(2') = 30plf$
$w_{S_2} = 25psf(2') = 50plf$



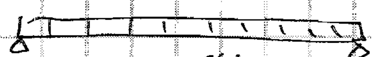
6'
4x8

② $w_D = 30plf$
$w_S = 50plf$



9.5'
4x6

③ $w_D = 15psf(19'/2) = 142.5plf$
$w_S = 25psf(19'/2) = 237.5plf$

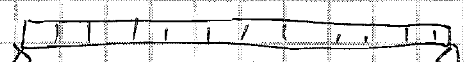


8'
4x8

④ $w_D = 15psf(32'/2) = 240plf$
$w_S = 25psf(32'/2) = 400plf$

6
4x8

⑤ $w_D = 15psf$
$w_L = 40psf$



20'7" clear span

Min. 15" 4x2 Floor Truss
@ 24" o.c.

FOR 80xx SE 20th
JOB
DATE 3-10-21
BY MM

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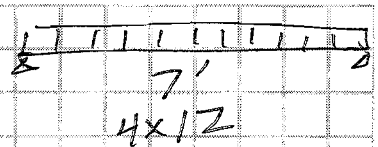
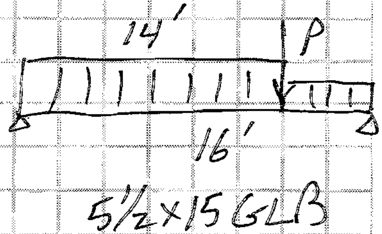
(6) $w_D = 15psf(28'/2) = 210plf$
$w_L = 40psf(28'/2) = 560plf$



(7) $w_{D_1} = 15psf(2' + 3'/2 + 21'/2) + 12psf(9') = 318plf$
$w_{L_1} = 40psf(21'/2) = 420plf$
$w_{S_1} = 25psf(2' + 3'/2) = 87.5plf$

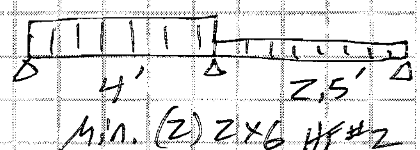
$w_{D_2} = 15psf(5'/2) = 37.5plf$
$w_{S_2} = 25psf(5'/2) = 62.5plf$

$P = 2300^{\#}DL + 200^{\#}LL + 3165^{\#}SL$



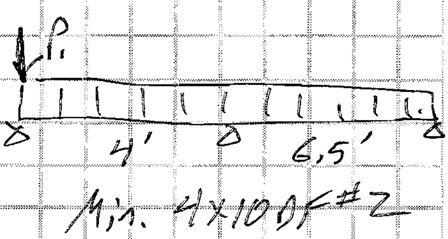
(8) $w_{D_1} = 15psf(19'/2) = 142.5plf$
$w_{L_1} = 40psf(19'/2) = 380plf$

$w_{D_2} = 15psf(10'/2) = 75plf$
$w_{L_2} = 40psf(10'/2) = 200plf$

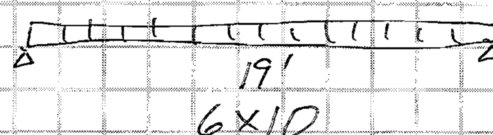


(9) $w_D = 15psf(31'/2 + 2'/2 + 10'/2) + 12psf(9') = 430.5plf$
$w_L = 40psf(10'/2) = 200plf$
$w_S = 25psf(31'/2 + 2'/2) = 412.5plf$

$P_1 = 2025^{\#}DL + 2975^{\#}SL$ from Girder abv.
$P_2 = \pm 940^{\#}WL \pm 1735^{\#}EL$



(10) $w_D = 15psf(6'/2) = 45plf$
$w_S = 25psf(6'/2) = 75plf$



FOR 80xx SE 20th
JOB
DATE 3-18-21
BY MM

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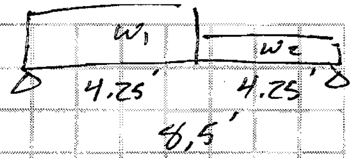
(11) $w_{D1} = 15psf(13'/2) = 97.5plf$
$w_{L1} = 40psf(13'/2) = 260plf$

$w_{D2} = 15psf(6'/2) = 45plf$
$w_{L2} = 40psf(6'/2) = 120plf$



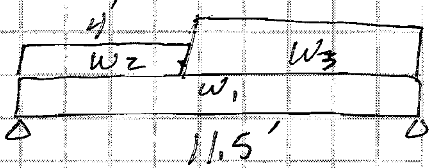
4x8 Min.

(12) $w_{D1} = 15psf(33'/2) + 12psf(9') = 355.5plf$
$w_{S1} = 25psf(33'/2) = 412.5plf$

$w_{D2} = 15psf(6'/2) = 45plf$
$w_{L2} = 40psf(6'/2) = 120plf$

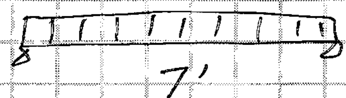
$w_{D3} = 15psf(18'/2) = 135plf$
$w_{L3} = 40psf(18'/2) = 360plf$

$P = 360^{\#}DL + 960^{\#}LL$ from (11)



3½x15 GLB

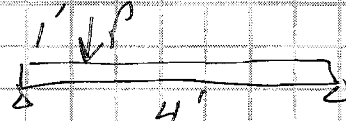
(13) $w_D = 15psf(23'/2 + 18'/2) + 12psf(9') = 415.5plf$
$w_L = 40psf(18'/2) = 360plf$
$w_S = 25psf(23'/2) = 287.5plf$



4x12 Min.

(14) $w_D = 15psf(2' + 1') + 12psf(9') = 153plf$
$w_L = 40plf$
$w_S = 50plf$

$P = 1460^{\#}DL + 1260^{\#}LL + 1010^{\#}SL$ from (13)



4x8

FOR 80xy SE 20th
JOB

DATE 3-18-21
BY [illegible]

MYERS ENGINEERING

Myers Engineering LLC
3206 50th St Ct NW, Ste 210-B
Gig Harbor, WA 98335
(253) 858-3248
Fax (253) 858-3249
myengineer@centurytel.net

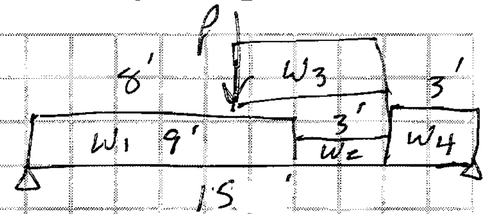
(15) $w_{D1} = 15psf\left(14'/2 + 2'\right) + 12psf(9') = 243plf$
$w_{L1} = 40psf\left(14'/2\right) = 280plf$
$w_{S1} = 25psf(2') = 50plf$

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

$w_{D3} = 15psf\left(12'/2\right) = 90plf$
$w_{S3} = 25psf\left(12'/2\right) = 150plf$

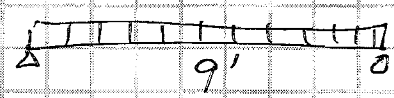
$w_{D4} = 15psf\left(17'/2\right)(1.25) = 160plf$
$w_{S4} = 25psf\left(17'/2\right)(1.25) = 266plf$

$P_1 = 410^{\#}DL + 600^{\#}SL$ from hip girder



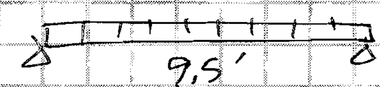
3½x15 GLB

(16) $w_D = 15psf\left(18'/2\right) = 135plf$
$w_S = 25psf\left(18'/2\right) = 225plf$



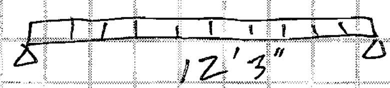
6x6

(17) $w_D = 15psf(2' + 1') + 12psf(9') = 153plf$
$w_L = 40plf$
$w_S = 50plf$



4x8

(18) $w_D = 15psf(2') = 30plf$
$w_S = 25psf(2') = 50plf$



4x6 Min

FOR 80xx SE 20th
JOB
DATE 3-18-21
BY MM

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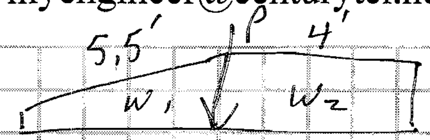
(19) $w_{D_1} = 15psf(11'/2) = 82.5plf$
$w_{S_1} = 25psf(11'/2) = 137.5plf$

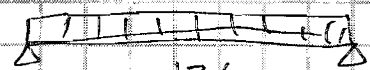
$w_{D_2} = 15psf(16'/2) = 120plf$
$w_{S_2} = 25psf(16'/2) = 200plf$

$P = 410^{\#}DL + 600^{\#}SL$ from girder



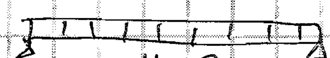
4x10

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



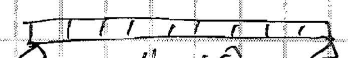
2x10 @ 16" o.c.

(21) $w_D = 15psf(30'/2 + 21'/2) = 382.5plf$
$w_L = 40psf(21'/2) + 30psf(30'/2) = 870plf$



4x10
5'4" clear span max

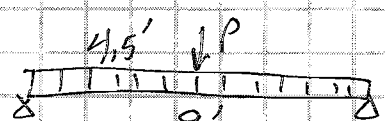
(22) $w_D = 15psf(22'/2) = 165plf$
$w_L = 40psf(22'/2) = 440plf$



4x10
7'8" clear span max

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

$P = 500^{\#}DL + 1350^{\#}LL$



(3) 2x10

FOR 80xx SE 20th
JOB
DATE 3-18-21
BY [initials]

MYERS ENGINEERING

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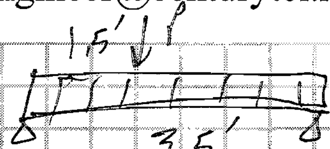

4x10

(24) $w_D = 15psf\left(\frac{32'}{2} + \frac{9'}{2}\right) + 12psf(9') = 415.5plf$

$w_L = 40psf\left(\frac{9'}{2}\right) = 180plf$

$w_S = 25psf\left(\frac{32'}{2}\right) = 400plf$

$P_1 = 1055\#DL + 1550\#SL$ from Girder Truss

$P_2 = 630\#DL + 1050\#SL$ from header

FOR 80xx SE 20th

JOB

DATE 3-18-21

BY

MYERS ENGINEERING

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Gig Harbor, WA 98335

Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 1. Header at Girder

CODE REFERENCES

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

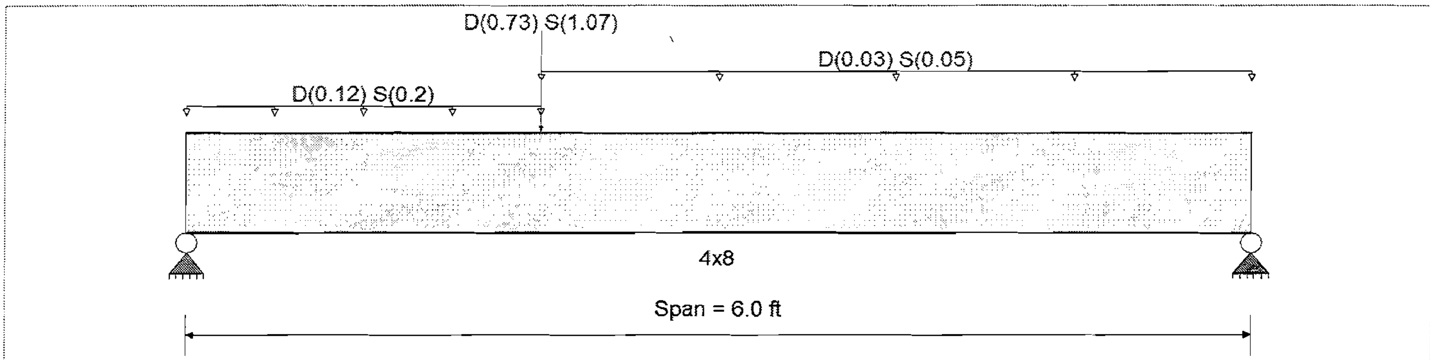
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

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

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

Fb +	900.0 psi	*E : Modulus of Elasticity*	
Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Fc - Perp	625.0 psi		
Fv	180.0 psi		
Ft	575.0 psi	Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.120, S = 0.20 k/ft, Extent = 0.0 -->> 2.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.030, S = 0.050 k/ft, Extent = 2.0 -->> 6.0 ft, Tributary Width = 1.0 ft
Point Load : D = 0.730, S = 1.070 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.882** 1	Maximum Shear Stress Ratio	=	**0.471** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,186.34 psi		=	97.58 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	1.993 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.055 in	Ratio =	1311 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.091 in	Ratio =	789 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	1.840	0.920
Overall MINimum	1.113	0.557
D Only	0.727	0.363
+D+L	0.727	0.363
+D+S	1.840	0.920
+D+0.750L	0.727	0.363
+D+0.750L+0.750S	1.562	0.781
+0.60D	0.436	0.218
S Only	1.113	0.557

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Wood Beam

Lic. # : KW-06008232

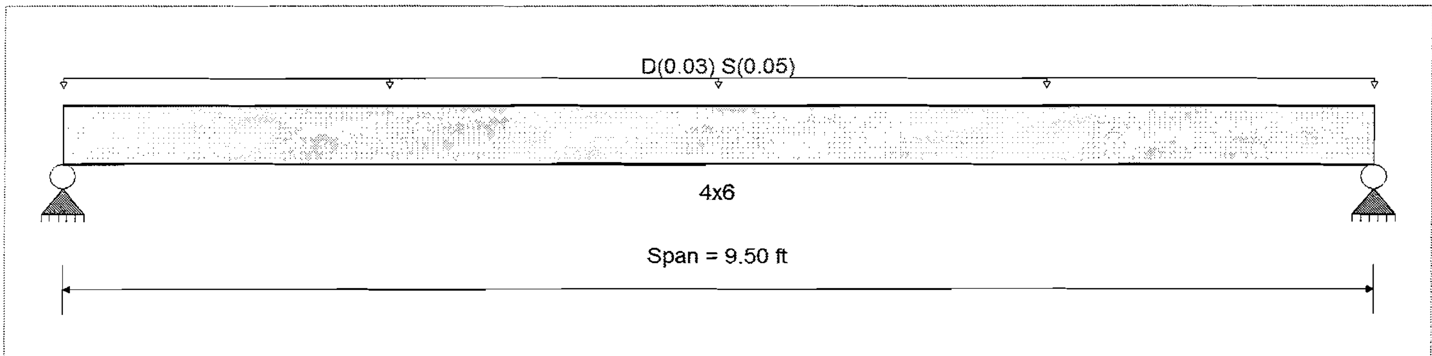
DESCRIPTION: 2. Header at Front Gable

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.030, S = 0.050 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.456: 1**	Maximum Shear Stress Ratio	=	**0.129 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
	=	613.74psi		=	26.80 psi
	=	1,345.50psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.750ft	Location of maximum on span	=	9.049 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.119 in	Ratio =	960 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.190 in	Ratio =	600 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.380	0.380
Overall MINimum	0.238	0.238
D Only	0.143	0.143
+D+L	0.143	0.143
+D+S	0.380	0.380
+D+0.750L	0.143	0.143
+D+0.750L+0.750S	0.321	0.321
+0.60D	0.086	0.086
S Only	0.238	0.238

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Gig Harbor, WA 98335

Wood Beam

DESCRIPTION: 3. Header at side Gable

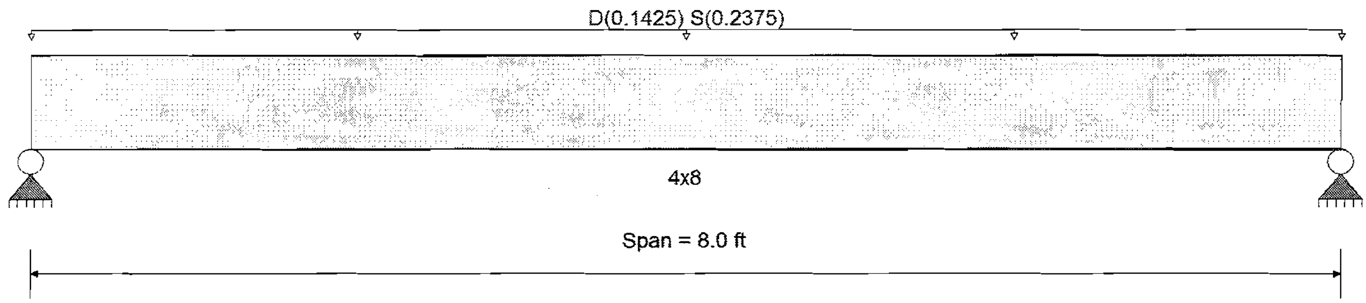
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1425, S = 0.2375 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.884 1**	Maximum Shear Stress Ratio	=	**0.371 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,189.77psi		=	76.74 psi
	=	1,345.50psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.000ft	Location of maximum on span	=	7.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.124 in	Ratio =	775 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.198 in	Ratio =	484 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.520	1.520
Overall MINimum	0.950	0.950
D Only	0.570	0.570
+D+L	0.570	0.570
+D+S	1.520	1.520
+D+0.750L	0.570	0.570
+D+0.750L+0.750S	1.283	1.283
+0.60D	0.342	0.342
S Only	0.950	0.950

MYERS ENGINEERING

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Gig Harbor, WA 98335

Wood Beam

DESCRIPTION: 4. Typical Roof Header

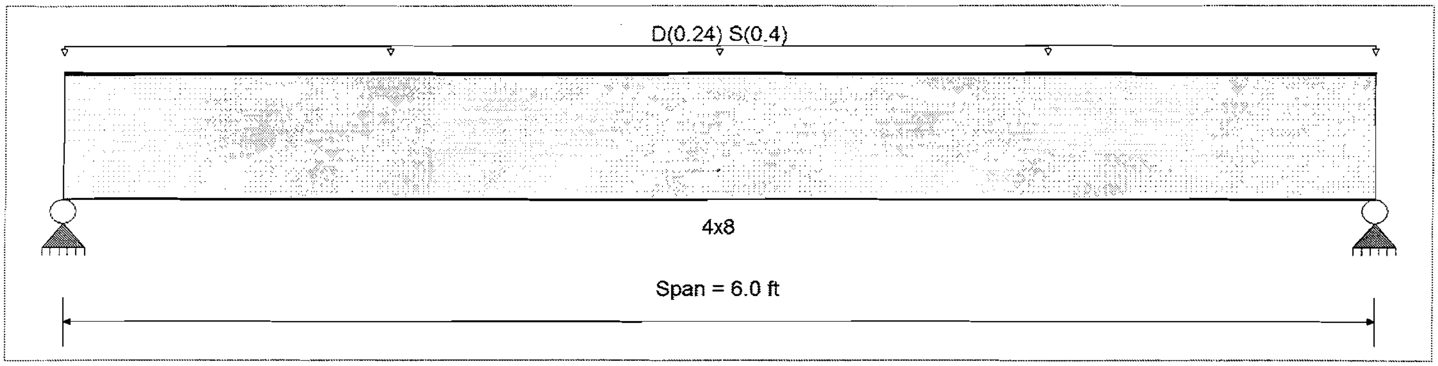
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

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

DESIGN SUMMARY

Design OK

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

Maximum Deflection

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

Vertical Reactions

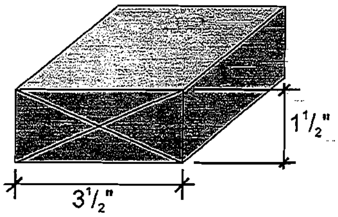
Support notation : Far left is #1 Values in KIPS

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

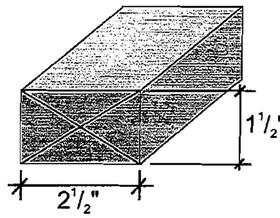
5

These allowable spans are based on NDS 91. Maximum deflection is limited by L/360 or L/480[1] under live load. Basic Lumber Design Values are $F_{(b)}$=2000 psi. $F_{(t)}$=1100 psi. $F_{(c)}$=2000 psi. E=1,800,000 psi. Duration Of Load = 1.00. Spacing of trusses are center to center (in inches). Top Chord Dead Load = 10 psf. Bottom Chord Dead Load = 5 psf. Center Line Chase = 24" max. Trusses must be designed for any special loading, such as concentrated loads. Other floor and roof loading conditions, a variety of species and other lumber grades are available.

4x2 Lumber



3x2 Lumber



40 PSF Live Load / 55 PSF Total Load

Center Spacing	Deflection Limit	4x2 12"	4x2 14"	4x2 16"	4x2 18"	4x2 20"	4x2 22"	3x2 12"	3x2 14"	3x2 16"	3x2 18"	3x2 20"	3x2 22"
16" o.c.	L/360	22'2"	24'11"	26'10"	28'8"	30'4"	31'11"	19'0"	20'9"	22'4"	23'10"	25'3"	26'7"
	L/480	20'2"	22'7"	24'11"	27'2"	29'4"	31'5"	18'0"	20'2"	22'4"	23'10"	25'3"	26'7"
19.2" o.c.	L/360	20'9"	22'8"	24'4"	26'0"	27'6"	29'0"	17'3"	18'9"	20'3"	21'7"	22'10"	24'1"
	L/480	18'11"	21'3"	23'6"	25'7"	27'6"	29'0"	16'11"	18'9"	20'3"	21'7"	22'10"	24'1"
24" o.c.	L/360	18'5"	20'1"	21'7"	23'1"	24'5"	25'9"	15'2"	16'7"	17'10"	19'1"	20'2"	21'3"
	L/480	17'7"	19'9"	21'7"	23'1"	24'5"	25'9"	15'2"	16'7"	17'10"	19'1"	20'2"	21'3"

60 PSF Live Load / 75 PSF Total Load

Center Spacing	Deflection Limit	4x2 12"	4x2 14"	4x2 16"	4x2 18"	4x2 20"	4x2 22"	3x2 12"	3x2 14"	3x2 16"	3x2 18"	3x2 20"	3x2 22"
16" o.c.	L/360	19'4"	21'4"	23'0"	24'6"	26'0"	27'4"	16'3"	17'9"	19'2"	20'5"	21'8"	22'9"
	L/480	17'7"	19'9"	21'10"	23'9"	25'8"	27'4"	15'9"	17'8"	19'2"	20'5"	21'8"	22'9"
19.2" o.c.	L/360	17'9"	19'4"	20'10"	22'3"	23'7"	24'10"	14'9"	16'1"	17'4"	18'6"	19'7"	20'7"
	L/480	16'7"	18'7"	20'6"	22'3"	23'7"	24'10"	14'9"	16'1"	17'4"	18'6"	19'7"	20'7"
24" o.c.	L/360	15'9"	17'2"	18'6"	19'9"	20'11"	22'0"	13'0"	14'2"	15'3"	16'4"	17'3"	18'2"
	L/480	15'4"	17'2"	18'6"	19'9"	20'11"	22'0"	13'0"	14'2"	15'3"	16'4"	17'3"	18'2"

85 PSF Live Load / 100 PSF Total Load

Center Spacing	Deflection Limit	4x2 12"	4x2 14"	4x2 16"	4x2 18"	4x2 20"	4x2 22"	3x2 12"	3x2 14"	3x2 16"	3x2 18"	3x2 20"	3x2 22"
16" o.c.	L/360	16'11"	18'6"	19'11"	21'3"	22'6"	23'8"	14'1"	15'5"	16'7"	17'8"	18'9"	19'9"
	L/480	15'8"	17'7"	19'5"	21'2"	22'6"	23'8"	14'0"	15'5"	16'7"	17'8"	18'9"	19'9"
19.2" o.c.	L/360	15'4"	16'9"	18'1"	19'3"	20'5"	21'6"	12'9"	13'11"	15'0"	16'0"	16'11"	17'10"
	L/480	14'9"	16'6"	18'1"	19'3"	20'5"	21'6"	12'9"	13'11"	15'0"	16'0"	16'11"	17'10"
24" o.c.	L/360	13'8"	14'10"	16'0"	17'1"	18'1"	19'1"	11'3"	12'3"	13'3"	14'1"	14'11"	15'9"
	L/480	13'8"	14'10"	16'0"	17'1"	18'1"	19'1"	11'3"	12'3"	13'3"	14'1"	14'11"	15'9"

(1) Vibration Control -- Research by Virginia Tech indicates that L/480 live load deflection criteria provides a high degree of resistance to floor vibration (bounce). The building designer desiring this benefit may choose to specify an L/480 live load deflection criteria to be used for the floor trusses.

40

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Wood Beam

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MYERS ENGINEERING

DESCRIPTION: 6. Floor beam at Foyer

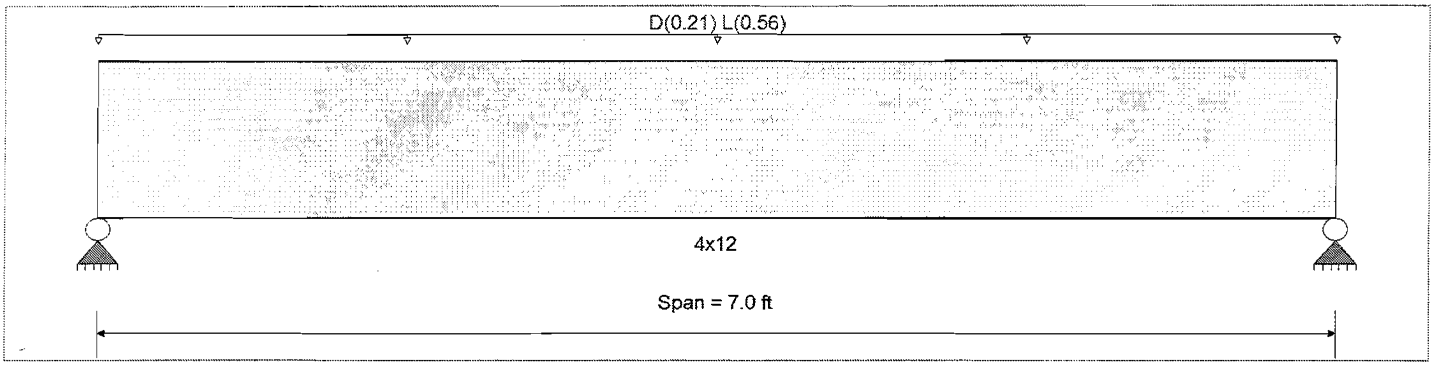
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.210, L = 0.560 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.774 1	Maximum Shear Stress Ratio	=	0.420 : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
	=	766.58psi		=	75.69 psi
	=	990.00psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.500ft	Location of maximum on span	=	6.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.046 in	Ratio =	1834 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.063 in	Ratio =	1333 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.695	2.695
Overall MINimum	1.960	1.960
D Only	0.735	0.735
+D+L	2.695	2.695
+D+S	0.735	0.735
+D+0.750L	2.205	2.205
+D+0.750L+0.750S	2.205	2.205
+0.60D	0.441	0.441
L Only	1.960	1.960
S Only		

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Wood Beam

File: 80xx SE 20th ST.ec6

Lic. # : KW-06008232 MYERS ENGINEERING

DESCRIPTION: 7. Garage Door Header

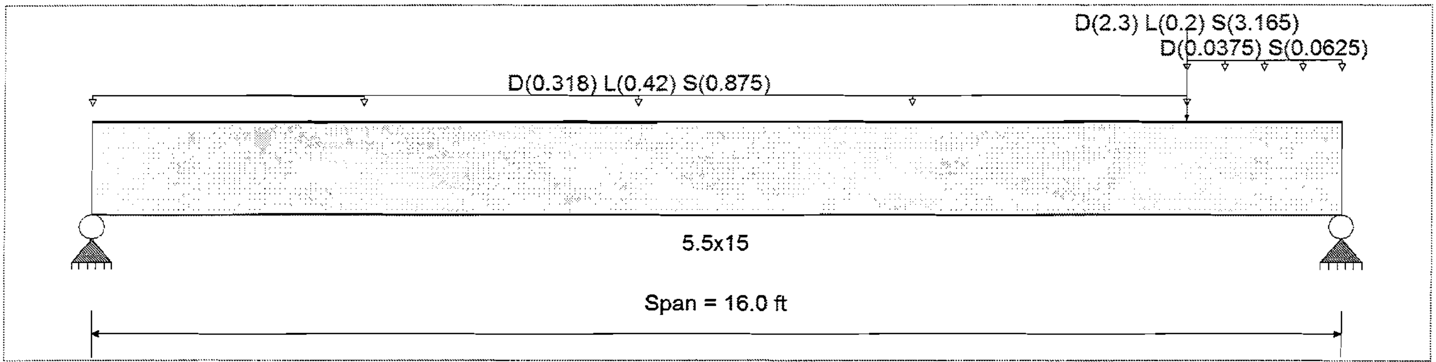
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.3180, L = 0.420, S = 0.8750 k/ft, Extent = 0.0 -->> 14.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.03750, S = 0.06250 k/ft, Extent = 14.0 -->> 16.0 ft, Tributary Width = 1.0 ft
Point Load : D = 2.30, L = 0.20, S = 3.165 k @ 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.950: 1	Maximum Shear Stress Ratio	=	0.726 : 1
Section used for this span		**5.5x15**	Section used for this span		**5.5x15**
	=	2,615.52 psi		=	221.31 psi
	=	2,753.98 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	8.350 ft	Location of maximum on span	=	14.774 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.512 in	Ratio =	375 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.757 in	Ratio =	253 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	10.766	12.277
Overall MINimum	7.294	8.246
D Only	2.796	4.031
+D+L	6.129	6.778
+D+S	10.091	12.277
+D+0.750L	5.296	6.091
+D+0.750L+0.750S	10.766	12.276
+0.60D	1.678	2.418
L Only	3.333	2.748
S Only	7.294	8.246

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Wood Beam

DESCRIPTION: 8. header at Shower

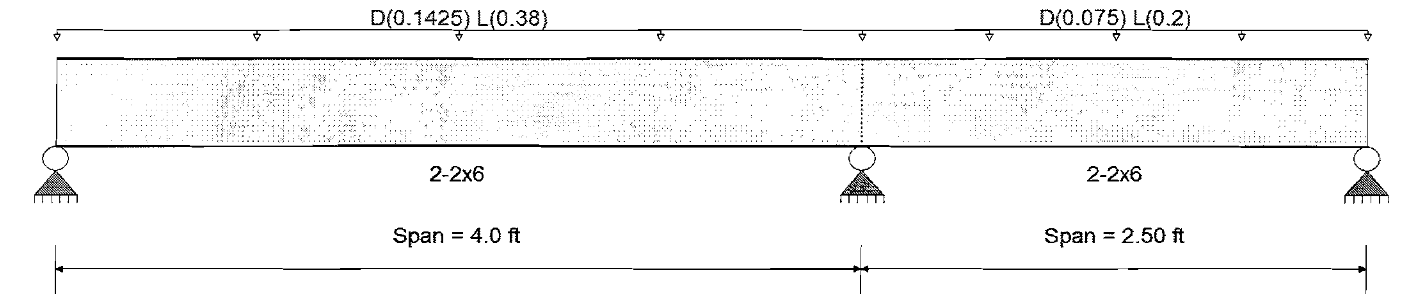
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840 pcf

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.1425, L = 0.380 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.0750, L = 0.20 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.521**: 1	Maximum Shear Stress Ratio	=	**0.602** : 1
Section used for this span		**2-2x6**	Section used for this span		**2-2x6**
	=	575.77 psi		=	90.26 psi
	=	1,105.00 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	3.553 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	1992 >=360
Max Upward Transient Deflection	-0.004 in	Ratio =	7979 >=360
Max Downward Total Deflection	0.033 in	Ratio =	1448 >=240
Max Upward Total Deflection	-0.005 in	Ratio =	5803 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.864	1.860	0.053
Overall MINimum	0.628	1.353	0.039
D Only	0.236	0.507	0.015
+D+L	0.864	1.860	0.053
+D+S	0.236	0.507	0.015
+D+0.750L	0.707	1.522	0.044
+D+0.750L+0.750S	0.707	1.522	0.044
+0.60D	0.141	0.304	0.009
L Only	0.628	1.353	0.039
S Only			

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Wood Beam

DESCRIPTION: 8a. header at Shower

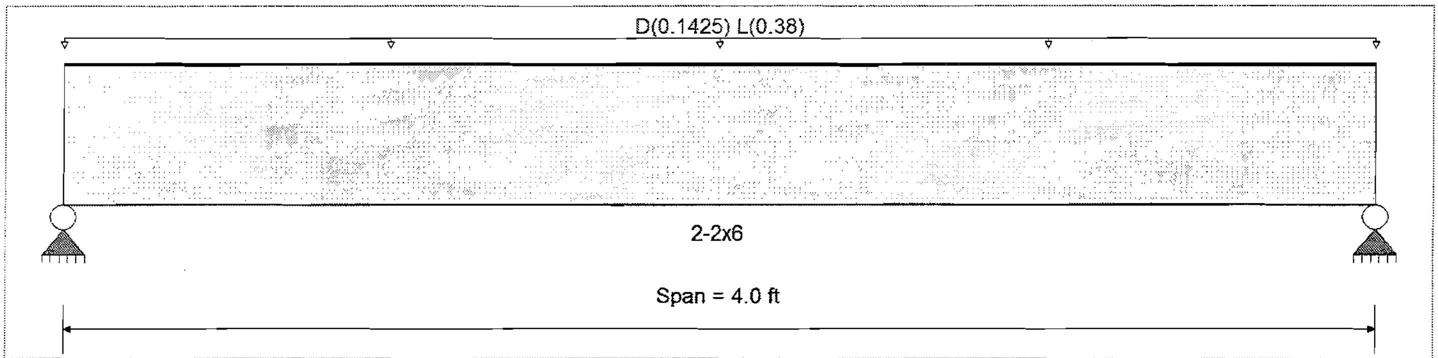
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850.0 psi	Ebend- xx	1,300.0ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840pcf

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



Applied Loads

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

Uniform Load : D = 0.1425, L = 0.380 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.750: 1	Maximum Shear Stress Ratio	=	0.490 : 1
Section used for this span		**2-2x6**	Section used for this span		**2-2x6**
	=	829.09psi		=	73.50 psi
	=	1,105.00psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.000ft	Location of maximum on span	=	3.547 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.041 in	Ratio =	1178 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.056 in	Ratio =	857 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.045	1.045
Overall MINimum	0.760	0.760
D Only	0.285	0.285
+D+L	1.045	1.045
+D+S	0.285	0.285
+D+0.750L	0.855	0.855
+D+0.750L+0.750S	0.855	0.855
+0.60D	0.171	0.171
L Only	0.760	0.760
S Only		

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Wood Beam

DESCRIPTION: 9. Front Rim Beam

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

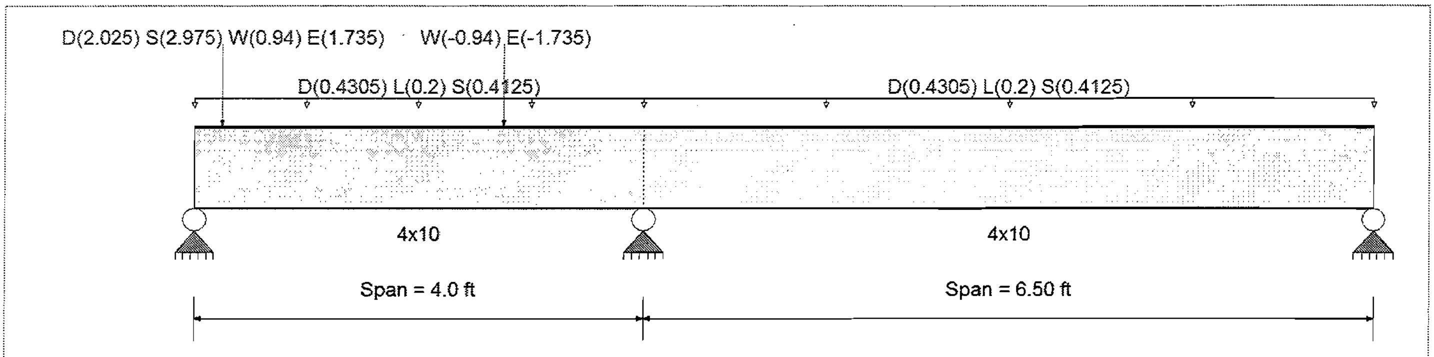
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

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

Fb +	900.0 psi	
Fb -	900.0 psi	
Fc - Prll	1,350.0 psi	
Fc - Perp	625.0 psi	
Fv	180.0 psi	
Ft	575.0 psi	

E : Modulus of Elasticity
Ebend- xx 1,600.0 ksi
Eminbend - xx 580.0 ksi

Density 31.210 pcf



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.4305, L = 0.20, S = 0.4125 , Tributary Width = 1.0 ft
- Point Load : D = 2.025, S = 2.975, W = 0.940, E = 1.735 k @ 0.250 ft
- Point Load : W = -0.940, E = -1.735 k @ 2.750 ft

Load for Span Number 2
- Uniform Load : D = 0.4305, L = 0.20, S = 0.4125 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.734** 1	Maximum Shear Stress Ratio	=	**0.692** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	911.02 psi		=	199.16 psi
	=	1,242.00 psi		=	288.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+1.105D+0.750L+0.750S-1.575E
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	3.240 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.023 in	Ratio =	3381 >=360
Max Upward Transient Deflection	-0.005 in	Ratio =	9339 >=360
Max Downward Total Deflection	0.053 in	Ratio =	1458 >=240
Max Upward Total Deflection	-0.005 in	Ratio =	10406 >=240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	5.465	7.122	2.341
Overall MINimum	-1.184	1.245	-0.061
D Only	2.302	3.126	1.117
+D+L	2.500	4.502	1.643
+D+S	5.465	6.206	2.180
+D+0.750L	2.450	4.158	1.512
+D+0.750L+0.750S	4.823	6.468	2.309
+D+0.60W	2.686	2.722	1.137
+D-0.60W	1.917	3.531	1.098
+D+0.70E	3.130	2.255	1.160

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DESCRIPTION: 9. Front Rim Beam

Vertical Reactions

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

Load Combination	Support 1	Support 2	Support 3
+D-0.70E	1.473	3.998	1.075
+D+0.750L+0.450W	2.739	3.855	1.527
+D+0.750L-0.450W	2.162	4.461	1.497
+D+0.750L+0.750S+0.450W	5.111	6.165	2.324
+D+0.750L+0.750S-0.450W	4.534	6.772	2.294
+D+0.750L+0.750S+0.5250E	5.444	5.815	2.341
+D+0.750L+0.750S-0.5250E	4.201	7.122	2.277
+0.60D+0.60W	1.766	1.471	0.690
+0.60D-0.60W	0.996	2.280	0.651
+0.60D+0.70E	2.209	1.005	0.713
+0.60D-0.70E	0.552	2.747	0.628
L Only	0.198	1.376	0.526
S Only	3.163	3.080	1.063
W Only	0.641	-0.674	0.033
-W	-0.641	0.674	-0.033
E Only	1.184	-1.245	0.061
E Only * -1.0	-1.184	1.245	-0.061

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Wood Beam

DESCRIPTION: 10. Roof beam in front of Garage

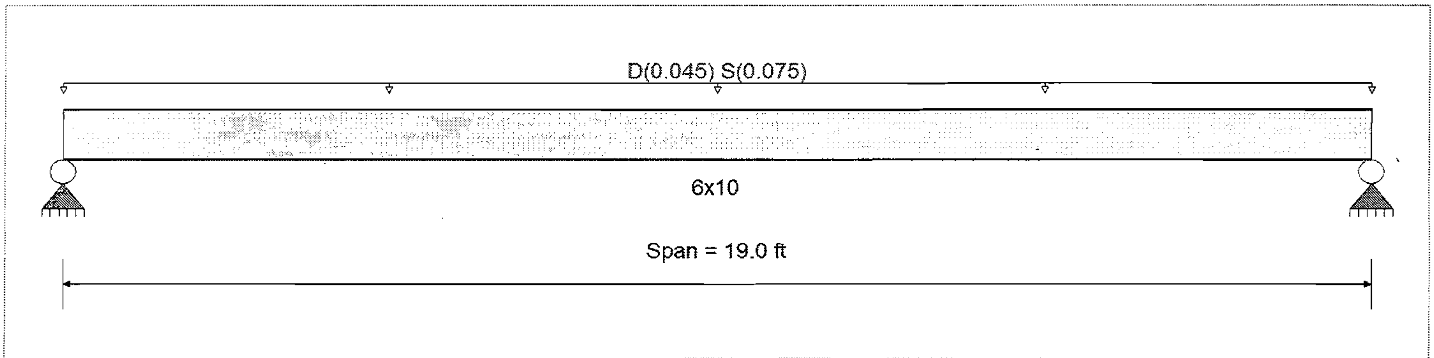
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.0450, S = 0.0750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.781**: 1	Maximum Shear Stress Ratio	=	**0.154** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	785.45psi		=	30.10 psi
	=	1,006.25psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.500ft	Location of maximum on span	=	18.237 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.433 in	Ratio =	526 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.693 in	Ratio =	329 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.140	1.140
Overall MINimum	0.713	0.713
D Only	0.428	0.428
+D+L	0.428	0.428
+D+S	1.140	1.140
+D+0.750L	0.428	0.428
+D+0.750L+0.750S	0.962	0.962
+0.60D	0.257	0.257
S Only	0.713	0.713

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DESCRIPTION: 11. Header at top of Stair

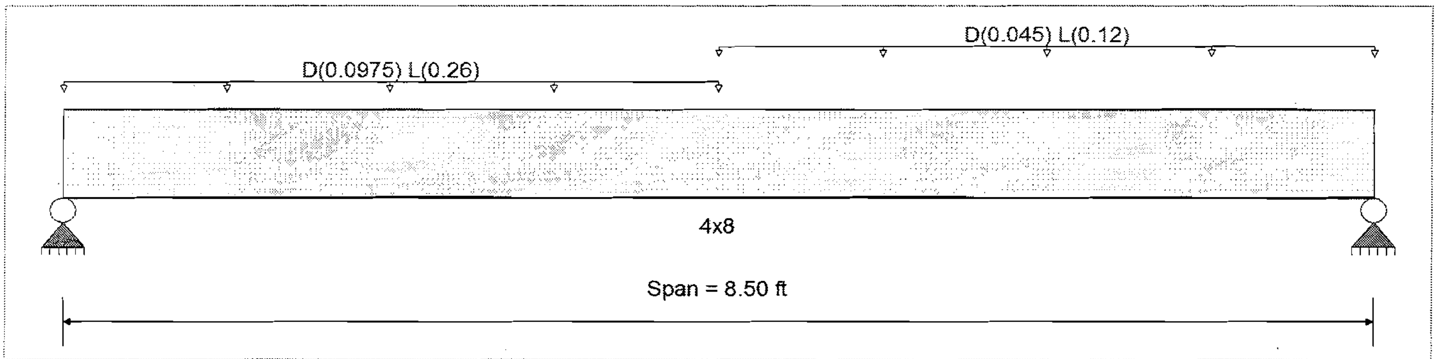
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.09750, L = 0.260 k/ft, Extent = 0.0 -->> 4.250 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.0450, L = 0.120 k/ft, Extent = 4.250 -->> 8.50 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.809 : 1**	Maximum Shear Stress Ratio	=	**0.363 : 1**
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	946.29 psi		=	65.27 psi
	=	1,170.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.692 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.126 in	Ratio =	807 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.174 in	Ratio =	586 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.315	0.906
Overall MINimum	0.956	0.659
D Only	0.359	0.247
+D+L	1.315	0.906
+D+S	0.359	0.247
+D+0.750L	1.076	0.741
+D+0.750L+0.750S	1.076	0.741
+0.60D	0.215	0.148
L Only	0.956	0.659
S Only		

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Wood Beam

DESCRIPTION: 12. Rim Beam at Great Rm

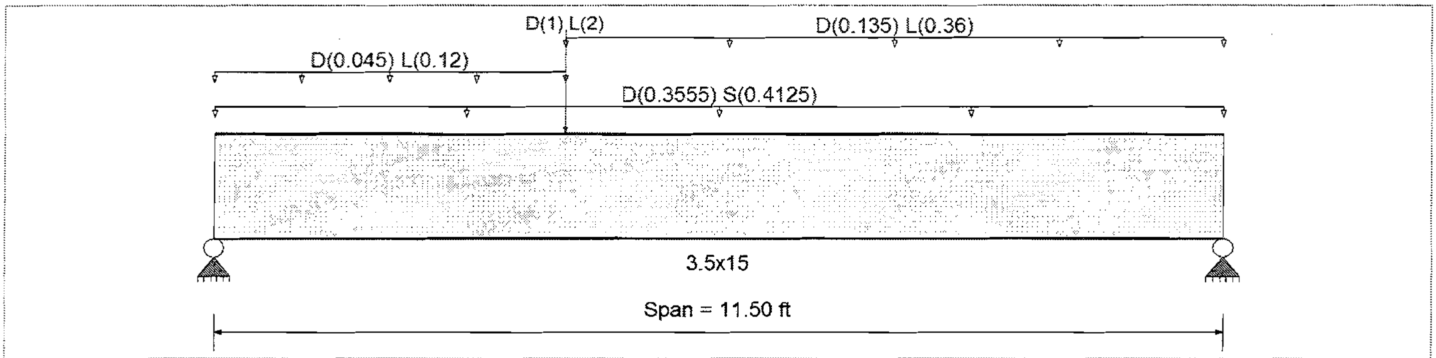
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.3555, S = 0.4125 , Tributary Width = 1.0 ft
Uniform Load : D = 0.0450, L = 0.120 k/ft, Extent = 0.0 -->> 4.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.1350, L = 0.360 k/ft, Extent = 4.0 -->> 11.50 ft, Tributary Width = 1.0 ft
Point Load : D = 1.0, L = 2.0 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.729 : 1**	Maximum Shear Stress Ratio	=	**0.555 : 1**
Section used for this span		**3.5x15**	Section used for this span		**3.5x15**
	=	1,748.96 psi		=	169.04 psi
	=	2,400.00 psi		=	304.75 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	4.785 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.121 in	Ratio =	1144 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.291 in	Ratio =	474 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.890	6.834
Overall MINimum	2.372	2.372
D Only	3.175	3.106
+D+L	5.756	5.704
+D+S	5.547	5.477
+D+0.750L	5.111	5.055
+D+0.750L+0.750S	6.890	6.834
+0.60D	1.905	1.863
L Only	2.581	2.599
S Only	2.372	2.372

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 13. Rim Beam at Pantry

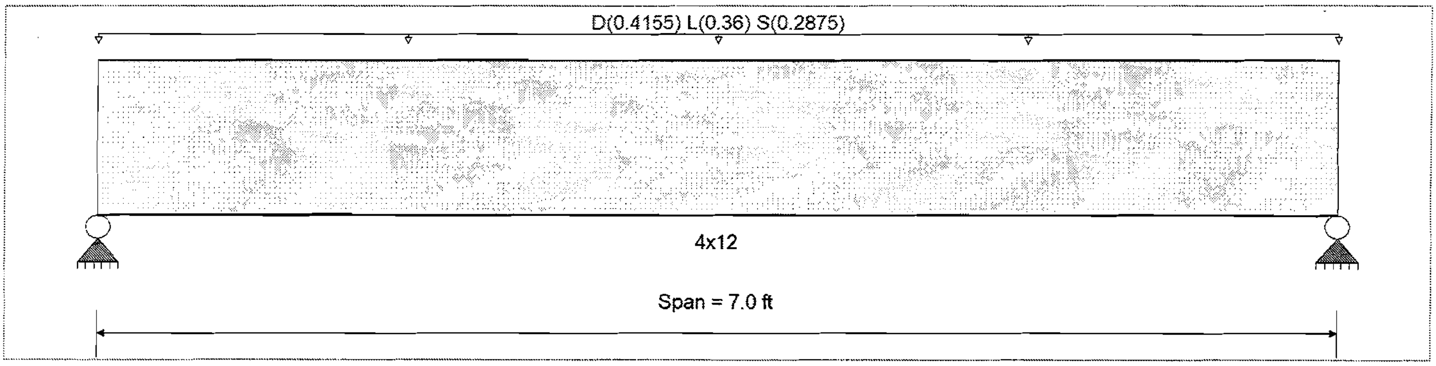
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.4155, L = 0.360, S = 0.2875 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.788** : 1	Maximum Shear Stress Ratio	=	**0.428** : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
	=	897.12 psi		=	88.58 psi
	=	1,138.50 psi		=	207.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.029 in	Ratio =	2853 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.074 in	Ratio =	1139 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.154	3.154
Overall MINimum	1.006	1.006
D Only	1.454	1.454
+D+L	2.714	2.714
+D+S	2.461	2.461
+D+0.750L	2.399	2.399
+D+0.750L+0.750S	3.154	3.154
+0.60D	0.873	0.873
L Only	1.260	1.260
S Only	1.006	1.006

MYERS ENGINEERING

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Wood Beam

DESCRIPTION: 14. Header at Pantry

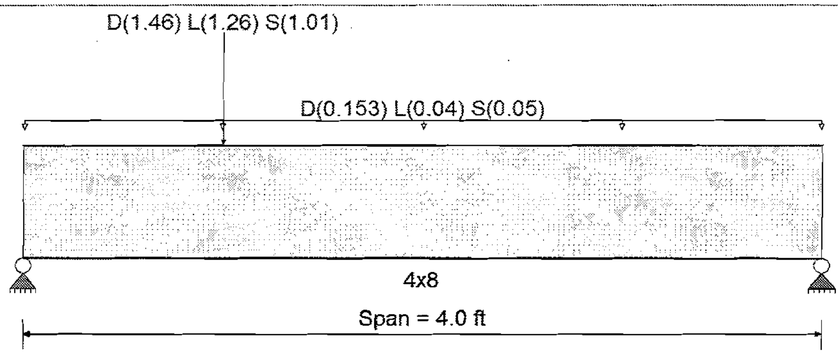
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.1530, L = 0.040, S = 0.050 , Tributary Width = 1.0 ft
Point Load : D = 1.460, L = 1.260, S = 1.010 k @ 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.785** : 1	Maximum Shear Stress Ratio	=	**0.766** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	1,056.10 psi		=	158.48 psi
	=	1,345.50 psi		=	207.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	1.007 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.013 in	Ratio =	3759 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.036 in	Ratio =	1336 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	2.813	1.232
Overall MINimum	0.858	0.353
D Only	1.401	0.671
+D+L	2.426	1.066
+D+S	2.259	1.024
+D+0.750L	2.170	0.967
+D+0.750L+0.750S	2.813	1.232
+0.60D	0.841	0.403
L Only	1.025	0.395
S Only	0.858	0.353

MYERS ENGINEERING

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Wood Beam

DESCRIPTION: 15. Rim beam over ADU

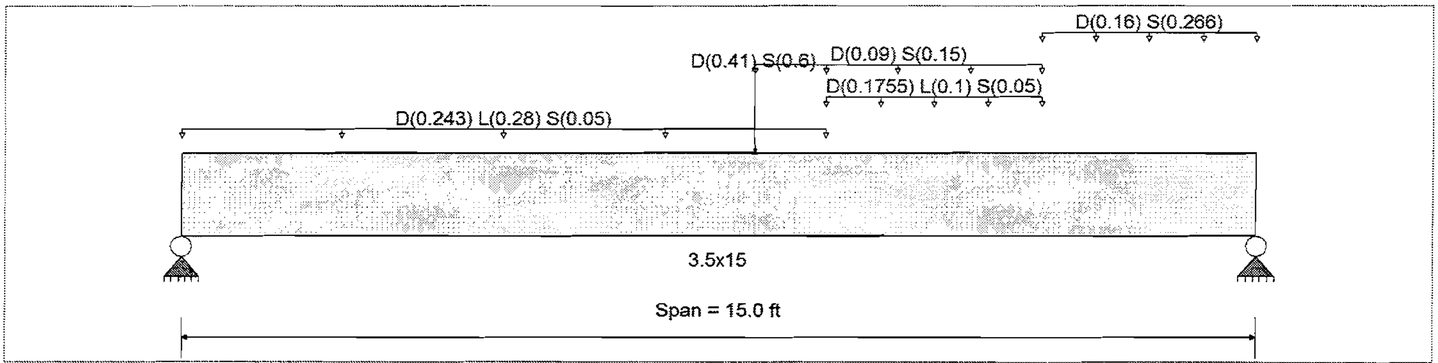
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
- Uniform Load : D = 0.2430, L = 0.280, S = 0.050 k/ft, Extent = 0.0 -->> 9.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.1755, L = 0.10, S = 0.050 k/ft, Extent = 9.0 -->> 12.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.090, S = 0.150 k/ft, Extent = 8.0 -->> 12.0 ft, Tributary Width = 1.0 ft
- Uniform Load : D = 0.160, S = 0.2660 k/ft, Extent = 12.0 -->> 15.0 ft, Tributary Width = 1.0 ft
- Point Load : D = 0.410, S = 0.60 k @ 8.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.574**: 1	Maximum Shear Stress Ratio	=	**0.353** : 1
Section used for this span		**3.5x15**	Section used for this span		**3.5x15**
	=	1,584.53 psi		=	93.49 psi
	=	2,760.00 psi		=	265.00 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+L
Location of maximum on span	=	7.993 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.135 in	Ratio =	1330 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.382 in	Ratio =	471 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.129	3.898
Overall MINimum	0.920	1.678
D Only	2.048	1.915
+D+L	3.902	2.881
+D+S	2.968	3.594
+D+0.750L	3.439	2.640
+D+0.750L+0.750S	4.129	3.898
+0.60D	1.229	1.149
L Only	1.854	0.966
S Only	0.920	1.678

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Wood Beam

File: 80xx SE 20th ST.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

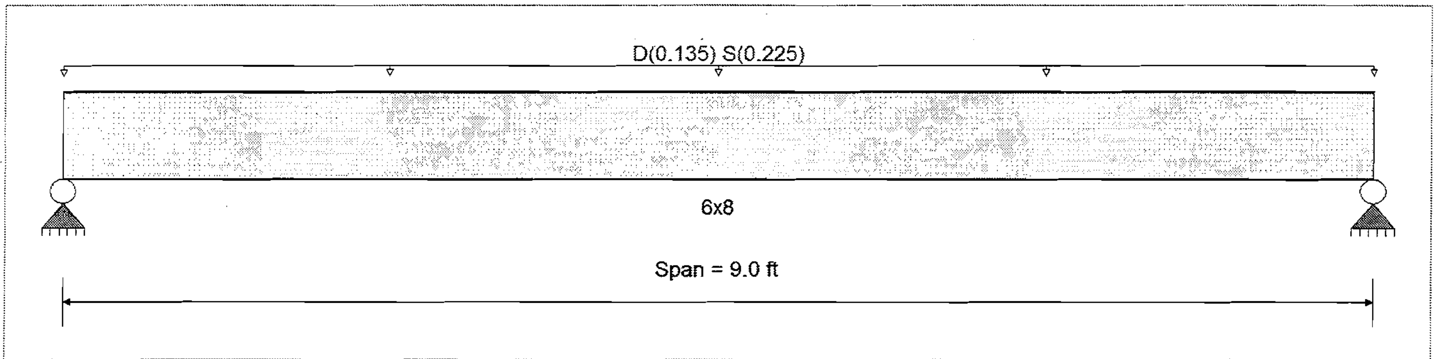
DESCRIPTION: 16. Roof beam at Cov'd Patio

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.1350, S = 0.2250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.843 : 1**	Maximum Shear Stress Ratio	=	**0.260 : 1**
Section used for this span		**6x8**	Section used for this span		**6x8**
	=	848.29 psi		=	50.74 psi
	=	1,006.25 psi		=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.376 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.133 in	Ratio =	812 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.213 in	Ratio =	507 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	1.620	1.620
Overall MINimum	1.013	1.013
D Only	0.608	0.608
+D+L	0.608	0.608
+D+S	1.620	1.620
+D+0.750L	0.608	0.608
+D+0.750L+0.750S	1.367	1.367
+0.60D	0.365	0.365
S Only	1.013	1.013

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Wood Beam

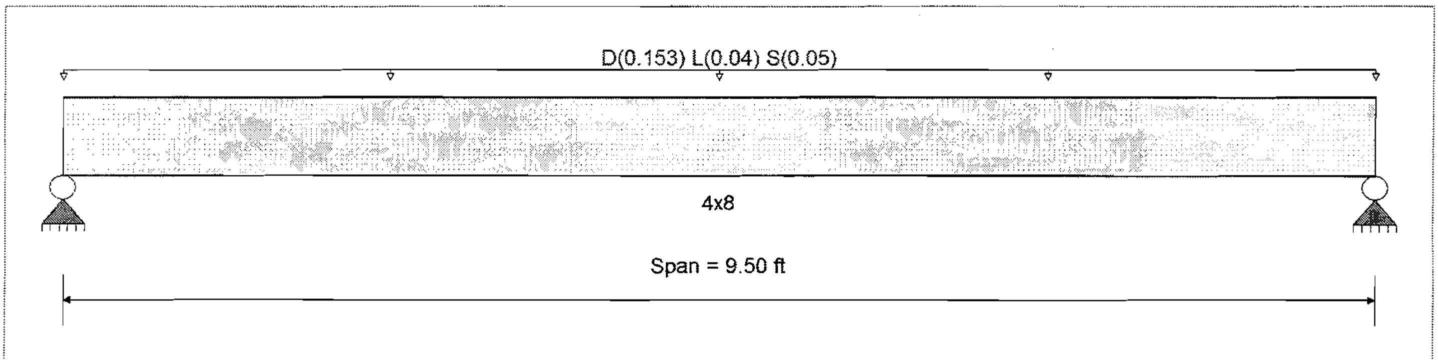
DESCRIPTION: 17. Header at Dining

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.1530, L = 0.040, S = 0.050 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.728** 1	Maximum Shear Stress Ratio	=	**0.264** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
	=	852.12 psi		=	47.47 psi
	=	1,170.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.750 ft	Location of maximum on span	=	8.911 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.052 in	Ratio =	2199 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.229 in	Ratio =	498 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.047	1.047
Overall MINimum	0.238	0.238
D Only	0.727	0.727
+D+L	0.917	0.917
+D+S	0.964	0.964
+D+0.750L	0.869	0.869
+D+0.750L+0.750S	1.047	1.047
+0.60D	0.436	0.436
L Only	0.190	0.190
S Only	0.238	0.238

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Wood Beam

DESCRIPTION: 18. Header at Nook SGD

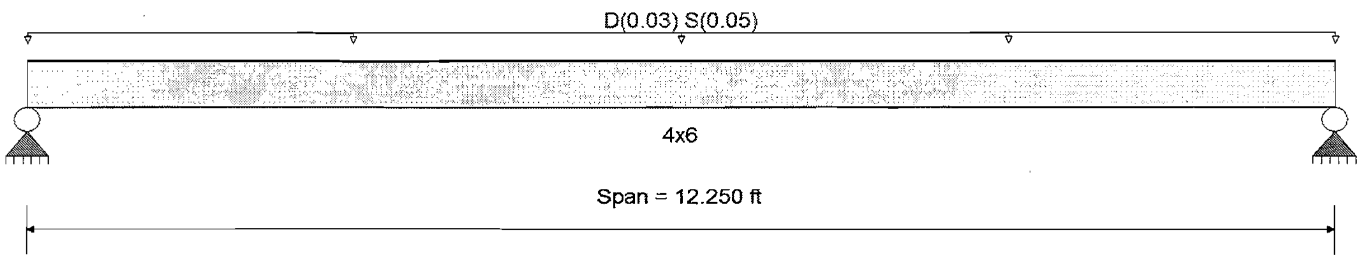
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.030, S = 0.050 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.758: 1**	Maximum Shear Stress Ratio	=	**0.171 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
	=	1,020.50psi		=	35.39 psi
	=	1,345.50psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.125ft	Location of maximum on span	=	11.803 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.328 in	Ratio =	447 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.525 in	Ratio =	279 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.490	0.490
Overall MINimum	0.306	0.306
D Only	0.184	0.184
+D+L	0.184	0.184
+D+S	0.490	0.490
+D+0.750L	0.184	0.184
+D+0.750L+0.750S	0.413	0.413
+0.60D	0.110	0.110
S Only	0.306	0.306

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Wood Beam

Lic. # : KW-06008232

DESCRIPTION: 19. Header at ADU

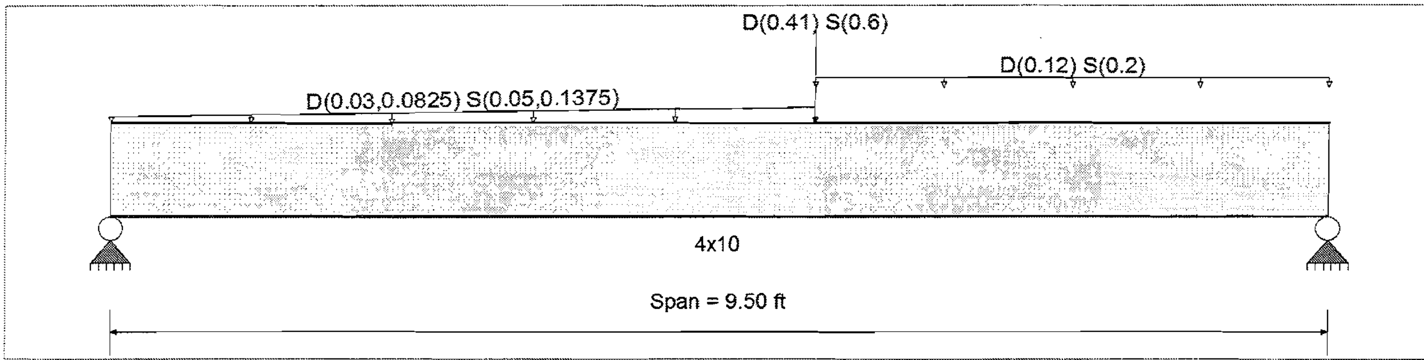
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Load for Span Number 1
Varying Uniform Load : D= 0.030->0.08250, S= 0.050->0.1375 k/ft, Extent = 0.0 -->> 5.50 ft, Trib Width = 1.0 ft

Uniform Load : D = 0.120, S = 0.20 k/ft, Extent = 5.50 -->> 9.50 ft, Tributary Width = 1.0 ft
Point Load : D = 0.410, S = 0.60 k @ 5.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.952** 1	Maximum Shear Stress Ratio	=	**0.364** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,182.25psi		=	75.39 psi
	=	1,242.00psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.513ft	Location of maximum on span	=	8.737 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.119 in	Ratio =	957 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.194 in	Ratio =	586 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.244	1.871
Overall MINimum	0.764	1.151
D Only	0.480	0.720
+D+L	0.480	0.720
+D+S	1.244	1.871
+D+0.750L	0.480	0.720
+D+0.750L+0.750S	1.053	1.583
+0.60D	0.288	0.432
S Only	0.764	1.151

MYERS ENGINEERING

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Wood Beam

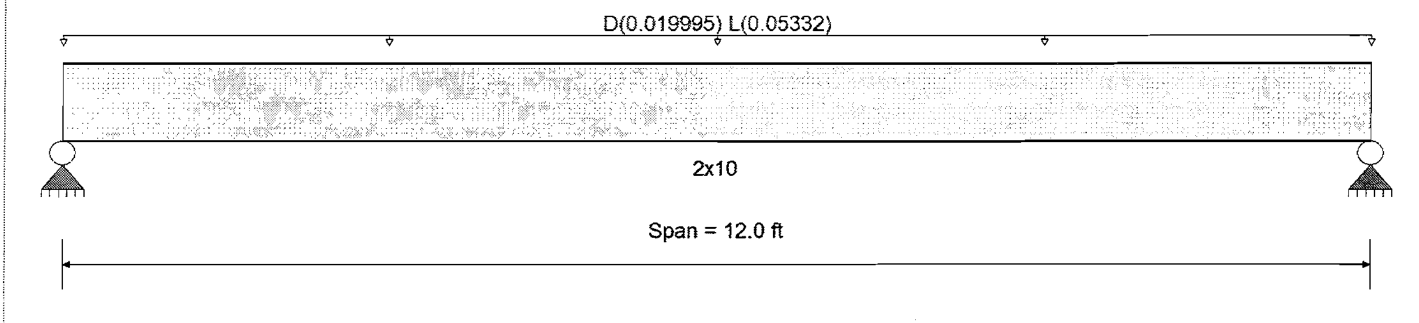
DESCRIPTION: 20. Floor Joist

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850 psi	Ebend- xx	1300 ksi
		Fc - Prll	1300 psi	Eminbend - xx	470 ksi
Wood Species	: Hem-Fir	Fc - Perp	405 psi		
Wood Grade	: No.2	Fv	150 psi		
		Ft	525 psi	Density	26.84 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.689: 1	Maximum Shear Stress Ratio	=	0.278 : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	740.33psi		=	41.65 psi
	=	1,075.25psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.195 in	Ratio =	740 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.268 in	Ratio =	538 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.440	0.440
Overall MINimum	0.320	0.320
D Only	0.120	0.120
+D+L	0.440	0.440
+D+S	0.120	0.120
+D+0.750L	0.360	0.360
+D+0.750L+0.750S	0.360	0.360
+0.60D	0.072	0.072
L Only	0.320	0.320
S Only		

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Wood Beam

DESCRIPTION: 21. Crawl Beam at Brg wall

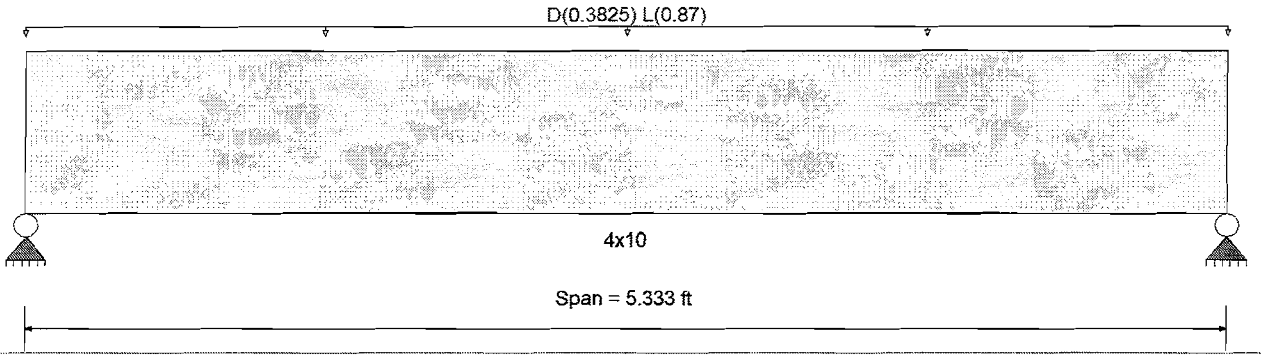
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

Uniform Load : D = 0.3825, L = 0.870 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.991**: 1	Maximum Shear Stress Ratio	=	**0.615** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,070.56 psi		=	110.69 psi
	=	1,080.00 psi		=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.667 ft	Location of maximum on span	=	4.574 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.043 in	Ratio =	1484 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.062 in	Ratio =	1030 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.340	3.340
Overall MINimum	2.320	2.320
D Only	1.020	1.020
+D+L	3.340	3.340
+D+S	1.020	1.020
+D+0.750L	2.760	2.760
+D+0.750L+0.750S	2.760	2.760
+0.60D	0.612	0.612
L Only	2.320	2.320
S Only		

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Wood Beam

DESCRIPTION: 22. Crawl Beam NOT at Brg wall

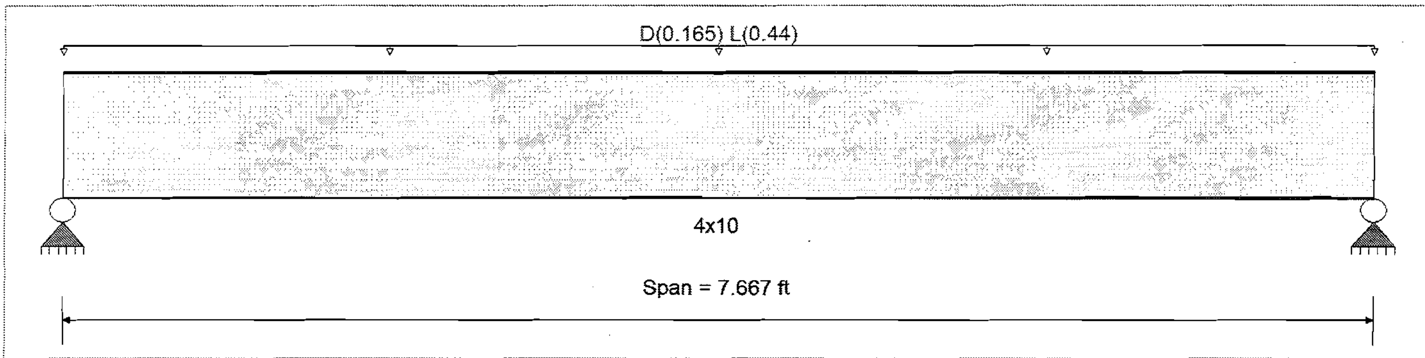
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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

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

DESIGN SUMMARY

Design OK

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

Maximum Deflection

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

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

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

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Wood Beam

File: 80xx SE 20th ST.ec6

Lic. # : KW-06008232

MYERS ENGINEERING

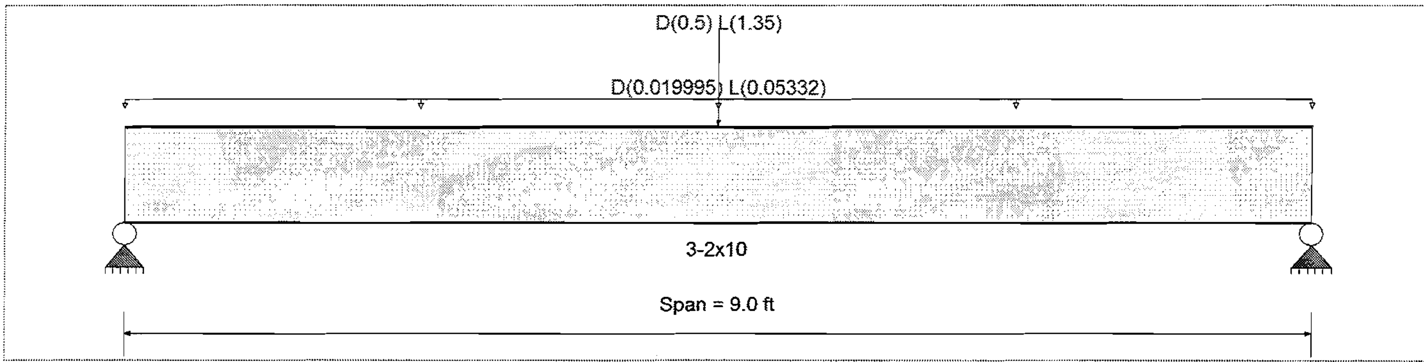
DESCRIPTION: 23. Joist at Stair

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft
Point Load : D = 0.50, L = 1.350 k @ 4.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.853** 1	Maximum Shear Stress Ratio	=	**0.288** : 1
Section used for this span		**3-2x10**	Section used for this span		**3-2x10**
	=	917.19 psi		=	43.23 psi
	=	1,075.25 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.245 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.113 in	Ratio =	957 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.155 in	Ratio =	697 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	1.255	1.255
Overall MINimum	0.915	0.915
D Only	0.340	0.340
+D+L	1.255	1.255
+D+S	0.340	0.340
+D+0.750L	1.026	1.026
+D+0.750L+0.750S	1.026	1.026
+0.60D	0.204	0.204
L Only	0.915	0.915
S Only		

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Wood Beam

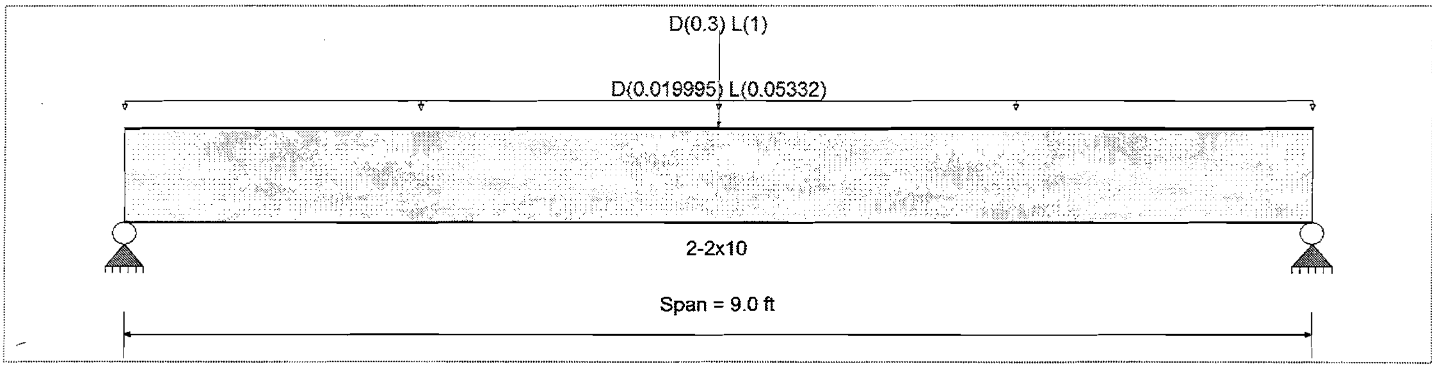
DESCRIPTION: 23a. Joist at Stair

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft
Point Load : D = 0.30, L = 1.0 k @ 4.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.957**: 1	Maximum Shear Stress Ratio	=	**0.333** : 1
Section used for this span		**2-2x10**	Section used for this span		**2-2x10**
	=	1,028.67 psi		=	49.97 psi
	=	1,075.25 psi		=	150.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.245 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.133 in	Ratio =	809 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.176 in	Ratio =	614 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	0.980	0.980
Overall MINimum	0.740	0.740
D Only	0.240	0.240
+D+L	0.980	0.980
+D+S	0.240	0.240
+D+0.750L	0.795	0.795
+D+0.750L+0.750S	0.795	0.795
+0.60D	0.144	0.144
L Only	0.740	0.740
S Only		

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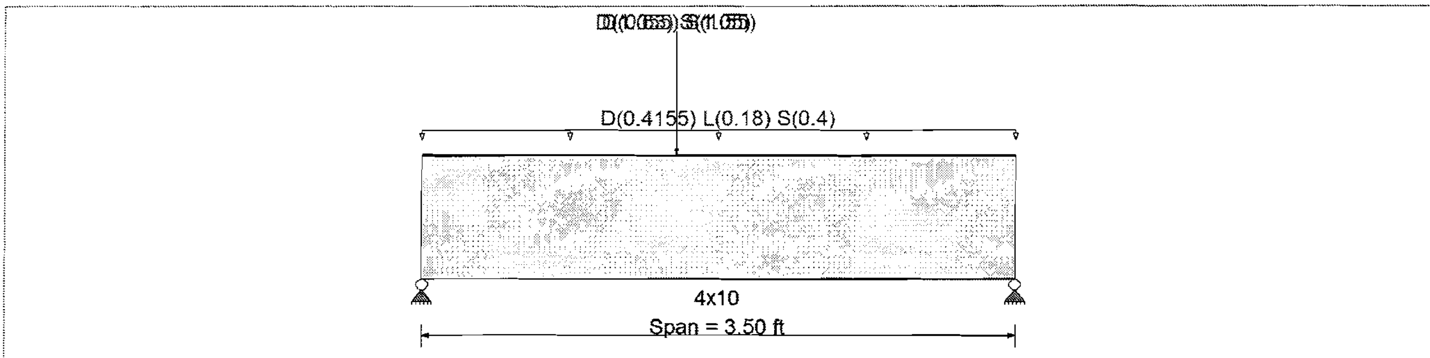
DESCRIPTION: 24. Header at Mud Rm

CODE REFERENCES

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

Material Properties

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



Applied Loads

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

Uniform Load : D = 0.4155, L = 0.180, S = 0.40 , Tributary Width = 1.0 ft
Point Load : D = 1.055, S = 1.550 k @ 1.50 ft
Point Load : D = 0.630, S = 1.050 k @ 1.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.945** : 1	Maximum Shear Stress Ratio	=	**0.728** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
	=	1,174.28 psi		=	150.61 psi
	=	1,242.00 psi		=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	1.507 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.014 in	Ratio =	2935 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.025 in	Ratio =	1678 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	3.876	3.264
Overall MINimum	2.186	1.814
D Only	1.690	1.449
+D+L	2.005	1.764
+D+S	3.876	3.264
+D+0.750L	1.926	1.686
+D+0.750L+0.750S	3.566	3.046
+0.60D	1.014	0.870
L Only	0.315	0.315
S Only	2.186	1.814

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Maximum Load For 6x6 DF#1 Wood Post

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

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

$E' := 1600000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.69$

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

Maximum Load For 6x6 HF#2 Treated Post

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

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

$E' := 1045000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

6x6 Treated Wood Post Properties

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

$h := 5.5 \cdot in$

$t := 5.5 \cdot in$

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

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

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

$C_p = 0.8$

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

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MYERS ENGINEERING

Concrete Beam

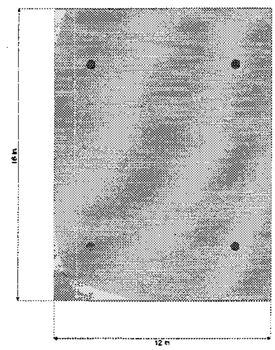
DESCRIPTION: A. Grade Beam at supporting Main & Upper

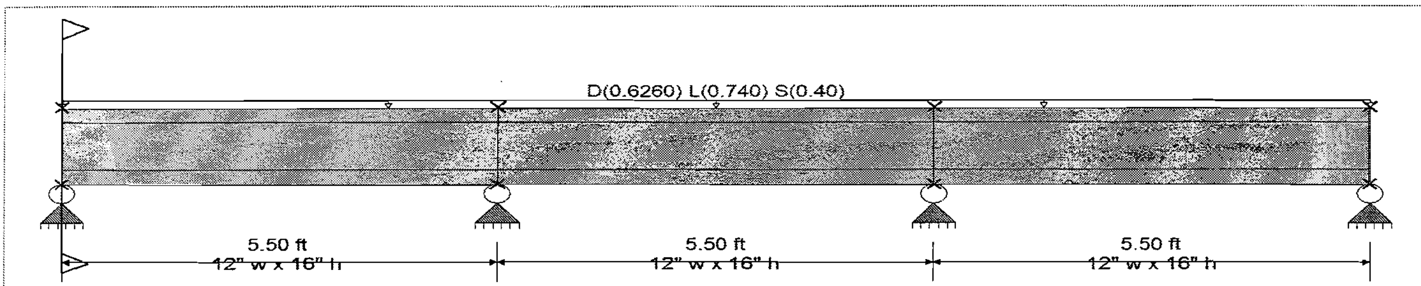
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c	= 2.50 ksi	ϕ Phi Values	Flexure :	0.90	
fr = $f'c^{1/2}$ * 7.50	= 375.0 psi		Shear :	0.750	
ψ Density	= 145.0 pcf	β_1	=	0.850	
λ LtWt Factor	= 1.0				
Elastic Modulus	= 2,850.0 ksi	Fy - Stirrups		40.0 ksi	
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi	
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3	
		Number of Resisting Legs Per Stirrup =		2	





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 5.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 5.50 ft in this span
Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 5.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 5.50 ft in this span
Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 5.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 5.50 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.6260, L = 0.740, S = 0.40
Uniform Load on ALL spans : D = 0.6260, L = 0.740, S = 0.40 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.288** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-7.161 k-ft	
Mn * Phi : Allowable	24.904 k-ft	
Location of maximum on span	0.000 ft	
Span # where maximum occurs	Span # 3	

Maximum Deflection			
Max Downward Transient Deflection	0.000 in	Ratio =	0 <720.0
Max Upward Transient Deflection	0.000 in	Ratio =	0 <720.0
Max Downward Total Deflection	0.002 in	Ratio =	42416 >=480.
Max Upward Total Deflection	0.000 in	Ratio =	0 <480.0

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	3.684	10.130	10.130	3.684

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Concrete Beam

DESCRIPTION: A. Grade Beam at supporting Main & Upper

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	0.880	2.420	2.420	0.880
D Only	1.803	4.957	4.957	1.803
+D+L	3.431	9.434	9.434	3.431
+D+S	2.683	7.377	7.377	2.683
+D+0.750L	3.024	8.315	8.315	3.024
+D+0.750L+0.750S	3.684	10.130	10.130	3.684
+0.60D	1.082	2.974	2.974	1.082
L Only	1.628	4.477	4.477	1.628
S Only	0.880	2.420	2.420	0.880

Shear Stirrup Requirements

Between 0.00 to 4.69 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 4.73 to 5.72 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 5.76 to 10.74 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 10.78 to 11.77 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 11.81 to 16.46 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.500	-6.88	24.90	0.28
Span # 2	2	5.500	-7.16	24.90	0.29
Span # 3	3	5.500	-7.16	24.90	0.29
+1.40D					
Span # 1	1	5.500	-3.33	24.90	0.13
Span # 2	2	5.500	-3.47	24.90	0.14
Span # 3	3	5.500	-3.47	24.90	0.14
+1.20D+1.60L					
Span # 1	1	5.500	-6.30	24.90	0.25
Span # 2	2	5.500	-6.56	24.90	0.26
Span # 3	3	5.500	-6.56	24.90	0.26
+1.20D+1.60L+0.50S					
Span # 1	1	5.500	-6.88	24.90	0.28
Span # 2	2	5.500	-7.16	24.90	0.29
Span # 3	3	5.500	-7.16	24.90	0.29
+1.20D+0.50L					
Span # 1	1	5.500	-3.93	24.90	0.16
Span # 2	2	5.500	-4.09	24.90	0.16
Span # 3	3	5.500	-4.09	24.90	0.16
+1.20D					
Span # 1	1	5.500	-2.86	24.90	0.11
Span # 2	2	5.500	-2.97	24.90	0.12
Span # 3	3	5.500	-2.97	24.90	0.12
+1.20D+0.50L+1.60S					
Span # 1	1	5.500	-5.79	24.90	0.23
Span # 2	2	5.500	-6.03	24.90	0.24
Span # 3	3	5.500	-6.03	24.90	0.24
+1.20D+1.60S					
Span # 1	1	5.500	-4.71	24.90	0.19
Span # 2	2	5.500	-4.91	24.90	0.20
Span # 3	3	5.500	-4.91	24.90	0.20
+1.20D+0.50L+0.50S					
Span # 1	1	5.500	-4.51	24.90	0.18
Span # 2	2	5.500	-4.70	24.90	0.19
Span # 3	3	5.500	-4.70	24.90	0.19
+1.384D+0.50L+0.70S					
Span # 1	1	5.500	-5.18	24.90	0.21
Span # 2	2	5.500	-5.40	24.90	0.22
Span # 3	3	5.500	-5.40	24.90	0.22
+0.90D					
Span # 1	1	5.500	-2.14	24.90	0.09
Span # 2	2	5.500	-2.23	24.90	0.09
Span # 3	3	5.500	-2.23	24.90	0.09
+0.7160D					
Span # 1	1	5.500	-1.70	24.90	0.07
Span # 2	2	5.500	-1.77	24.90	0.07

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Concrete Beam

DESCRIPTION: A. Grade Beam at supporting Main & Upper

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 3	3	5.500	-1.77	24.90	0.07

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+0.750L+0.750S	1	0.0016	2.530	+D+0.750L+0.750S	-0.0000	5.610
+D+0.750L+0.750S	2	0.0001	2.750	+D+0.750L+0.750S	-0.0001	4.950
+D+0.750L+0.750S	3	0.0016	2.970		0.0000	4.950

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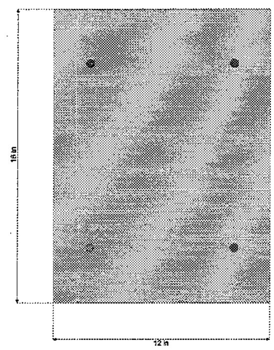
DESCRIPTION: B. Grade Beam supporting Great Rm Room

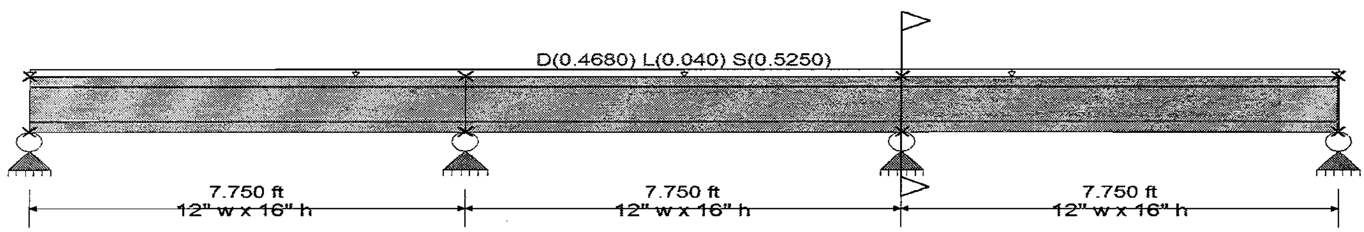
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c	= 2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} * 7.50$	= 375.0 psi		Shear :	0.750
ψ Density	= 145.0 pcf	β_1	=	0.850
λ LtWt Factor	= 1.0			
Elastic Modulus	= 3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3
		Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in

Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.750 ft in this span
2-#4 at 3.0 in from Bottom, from 0.0 to 7.750 ft in this span

Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.750 ft in this span
2-#4 at 3.0 in from Bottom, from 0.0 to 7.750 ft in this span

Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.750 ft in this span
2-#4 at 3.0 in from Bottom, from 0.0 to 7.750 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.4680, L = 0.040, S = 0.5250
Uniform Load on ALL spans : D = 0.4680, L = 0.040, S = 0.5250 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.399** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-9.932	k-ft
Mn * Phi : Allowable	24.904	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 2	

Maximum Deflection				
Max Downward Transient Deflection	0.002 in	Ratio =	52964	>=720.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<720.C
Max Downward Total Deflection	0.004 in	Ratio =	23438	>=480.
Max Upward Total Deflection	0.000 in	Ratio =	0	<480.C

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	510.90	510.90

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	3.678	10.113	10.113	3.678

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Concrete Beam

DESCRIPTION: B. Grade Beam supporting Great Rm Room

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	0.124	0.341	0.341	0.124
D Only	2.050	5.638	5.638	2.050
+D+L	2.174	5.979	5.979	2.174
+D+S	3.678	10.113	10.113	3.678
+D+0.750L	2.143	5.894	5.894	2.143
+D+0.750L+0.750S	3.364	9.250	9.250	3.364
+0.60D	1.230	3.383	3.383	1.230
L Only	0.124	0.341	0.341	0.124
S Only	1.628	4.476	4.476	1.628

Shear Stirrup Requirements

Between 0.00 to 6.67 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 6.72 to 8.06 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 8.11 to 15.14 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 15.19 to 16.53 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 16.59 to 23.20 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	7.750	-9.54	24.90	0.38
Span # 2	2	7.750	-9.93	24.90	0.40
Span # 3	3	7.750	-9.93	24.90	0.40
+1.40D					
Span # 1	1	7.750	-5.34	24.90	0.21
Span # 2	2	7.750	-5.56	24.90	0.22
Span # 3	3	7.750	-5.56	24.90	0.22
+1.20D+1.60L					
Span # 1	1	7.750	-4.95	24.90	0.20
Span # 2	2	7.750	-5.15	24.90	0.21
Span # 3	3	7.750	-5.15	24.90	0.21
+1.20D+1.60L+0.50S					
Span # 1	1	7.750	-6.46	24.90	0.26
Span # 2	2	7.750	-6.73	24.90	0.27
Span # 3	3	7.750	-6.73	24.90	0.27
+1.20D+0.50L					
Span # 1	1	7.750	-4.69	24.90	0.19
Span # 2	2	7.750	-4.89	24.90	0.20
Span # 3	3	7.750	-4.89	24.90	0.20
+1.20D					
Span # 1	1	7.750	-4.58	24.90	0.18
Span # 2	2	7.750	-4.77	24.90	0.19
Span # 3	3	7.750	-4.77	24.90	0.19
+1.20D+0.50L+1.60S					
Span # 1	1	7.750	-9.54	24.90	0.38
Span # 2	2	7.750	-9.93	24.90	0.40
Span # 3	3	7.750	-9.93	24.90	0.40
+1.20D+1.60S					
Span # 1	1	7.750	-9.42	24.90	0.38
Span # 2	2	7.750	-9.81	24.90	0.39
Span # 3	3	7.750	-9.81	24.90	0.39
+1.20D+0.50L+0.50S					
Span # 1	1	7.750	-6.21	24.90	0.25
Span # 2	2	7.750	-6.46	24.90	0.26
Span # 3	3	7.750	-6.46	24.90	0.26
+1.384D+0.50L+0.70S					
Span # 1	1	7.750	-7.51	24.90	0.30
Span # 2	2	7.750	-7.82	24.90	0.31
Span # 3	3	7.750	-7.82	24.90	0.31
+0.90D					
Span # 1	1	7.750	-3.43	24.90	0.14
Span # 2	2	7.750	-3.57	24.90	0.14
Span # 3	3	7.750	-3.57	24.90	0.14
+0.7160D					
Span # 1	1	7.750	-2.73	24.90	0.11
Span # 2	2	7.750	-2.84	24.90	0.11

MYERS ENGINEERING

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DESCRIPTION: B. Grade Beam supporting Great Rm Room

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 3	3	7.750	-2.84	24.90	0.11

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+S	1	0.0040	3.565	+D+S	-0.0001	7.905
+D+S	2	0.0003	3.875	+D+S	-0.0002	0.775
+D+S	3	0.0040	4.185		0.0000	0.775

MYERS ENGINEERING

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Concrete Beam

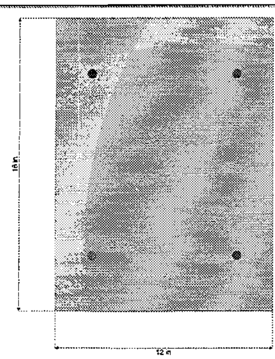
DESCRIPTION: C. Grade Beam supporting Gable Walls

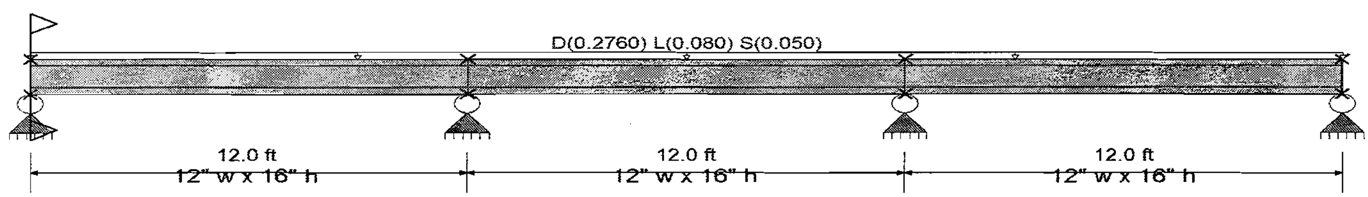
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 12.0 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 12.0 ft in this span
Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 12.0 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 12.0 ft in this span
Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 12.0 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 12.0 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.2760, L = 0.080, S = 0.050
Uniform Load on ALL spans : D = 0.2760, L = 0.080, S = 0.050 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.419** : 1	Maximum Deflection		
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.002 in	Ratio = 93630 >=720.
Mu : Applied	-10.434 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <720.C
Mn * Phi : Allowable	24.904 k-ft	Max Downward Total Deflection	0.011 in	Ratio = 13214 >=480.
		Max Upward Total Deflection	0.000 in	Ratio = 0 <480.C
Location of maximum on span	0.000 ft			
Span # where maximum occurs	Span # 2			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	2.721	7.482	7.482	2.721

MYERS ENGINEERING

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DESCRIPTION: C. Grade Beam supporting Gable Walls

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	0.240	0.660	0.660	0.240
D Only	2.253	6.195	6.195	2.253
+D+L	2.637	7.251	7.251	2.637
+D+S	2.493	6.855	6.855	2.493
+D+0.750L	2.541	6.987	6.987	2.541
+D+0.750L+0.750S	2.721	7.482	7.482	2.721
+0.60D	1.352	3.717	3.717	1.352
L Only	0.384	1.056	1.056	0.384
S Only	0.240	0.660	0.660	0.240

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	12.000	-10.02	24.90	0.40
Span # 2	2	12.000	-10.43	24.90	0.42
Span # 3	3	12.000	-10.43	24.90	0.42
+1.40D					
Span # 1	1	12.000	-9.09	24.90	0.36
Span # 2	2	12.000	-9.46	24.90	0.38
Span # 3	3	12.000	-9.46	24.90	0.38
+1.20D+1.60L					
Span # 1	1	12.000	-9.56	24.90	0.38
Span # 2	2	12.000	-9.95	24.90	0.40
Span # 3	3	12.000	-9.95	24.90	0.40
+1.20D+1.60L+0.50S					
Span # 1	1	12.000	-9.90	24.90	0.40
Span # 2	2	12.000	-10.31	24.90	0.41
Span # 3	3	12.000	-10.31	24.90	0.41
+1.20D+0.50L					
Span # 1	1	12.000	-8.34	24.90	0.33
Span # 2	2	12.000	-8.69	24.90	0.35
Span # 3	3	12.000	-8.69	24.90	0.35
+1.20D					
Span # 1	1	12.000	-7.79	24.90	0.31
Span # 2	2	12.000	-8.11	24.90	0.33
Span # 3	3	12.000	-8.11	24.90	0.33
+1.20D+0.50L+1.60S					
Span # 1	1	12.000	-9.45	24.90	0.38
Span # 2	2	12.000	-9.84	24.90	0.40
Span # 3	3	12.000	-9.84	24.90	0.40
+1.20D+1.60S					
Span # 1	1	12.000	-8.89	24.90	0.36
Span # 2	2	12.000	-9.26	24.90	0.37
Span # 3	3	12.000	-9.26	24.90	0.37
+1.20D+0.50L+0.50S					
Span # 1	1	12.000	-8.69	24.90	0.35
Span # 2	2	12.000	-9.05	24.90	0.36
Span # 3	3	12.000	-9.05	24.90	0.36
+1.384D+0.50L+0.70S					
Span # 1	1	12.000	-10.02	24.90	0.40
Span # 2	2	12.000	-10.43	24.90	0.42
Span # 3	3	12.000	-10.43	24.90	0.42
+0.90D					
Span # 1	1	12.000	-5.84	24.90	0.23
Span # 2	2	12.000	-6.08	24.90	0.24
Span # 3	3	12.000	-6.08	24.90	0.24
+0.7160D					
Span # 1	1	12.000	-4.65	24.90	0.19
Span # 2	2	12.000	-4.84	24.90	0.19
Span # 3	3	12.000	-4.84	24.90	0.19

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DESCRIPTION: C. Grade Beam supporting Gable Walls

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+0.750L+0.750S	1	0.0109	5.520	+D+0.750L+0.750S	-0.0002	12.240
+D+0.750L+0.750S	2	0.0008	6.000	+D+0.750L+0.750S	-0.0006	1.200
+D+0.750L+0.750S	3	0.0109	6.480		0.0000	1.200

MYERS ENGINEERING

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Concrete Beam

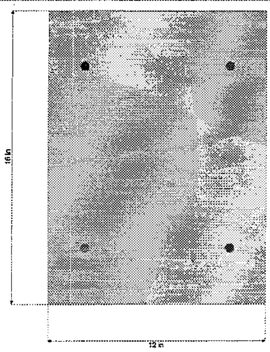
DESCRIPTION: D. Grade Beam supporting Great Rm Floor

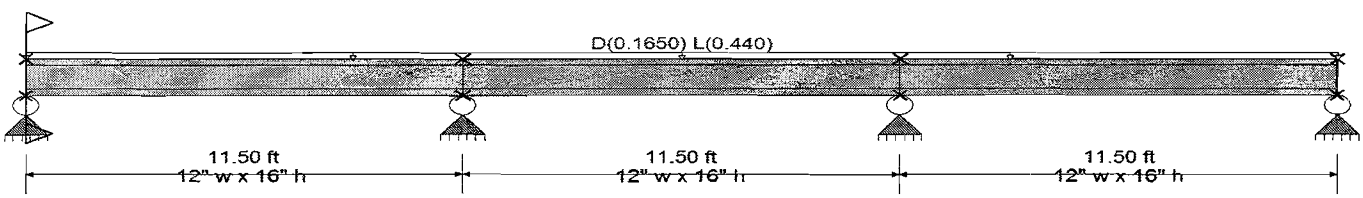
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c = 2.50 ksi		ϕ Phi Values	Flexure :	0.90
$fr = f'c^{1/2} * 7.50$ = 375.0 psi			Shear :	0.750
ψ Density = 145.0 pcf		β_1	=	0.850
λ LtWt Factor = 1.0				
Elastic Modulus = 3,122.0 ksi		Fy - Stirrups		40.0 ksi
fy - Main Rebar = 60.0 ksi		E - Stirrups	=	29,000.0 ksi
E - Main Rebar = 29,000.0 ksi		Stirrup Bar Size #		3
		Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 11.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 11.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 11.50 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.1650, L = 0.440
Uniform Load on ALL spans : D = 0.1650, L = 0.440 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.602** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.007 in	Ratio =	19342 >=720
Mu : Applied	-14.997 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <720.0
Mn * Phi : Allowable	24.904 k-ft	Max Downward Total Deflection	0.013 in	Ratio =	10660 >=480
Location of maximum on span	0.000 ft	Max Upward Total Deflection	0.000 in	Ratio =	0 <480.0
Span # where maximum occurs	Span # 2				

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	3.672	10.099	10.099	3.672

MYERS ENGINEERING

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DESCRIPTION: D. Grade Beam supporting Great Rm Floor

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	0.989	2.720	2.720	0.989
D Only	1.648	4.533	4.533	1.648
+D+L	3.672	10.099	10.099	3.672
+D+0.750L	3.166	8.707	8.707	3.166
+0.60D	0.989	2.720	2.720	0.989
L Only	2.024	5.566	5.566	2.024

Shear Stirrup Requirements

Between 0.00 to 9.81 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 9.89 to 12.11 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 12.19 to 22.31 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 22.39 to 24.61 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 24.69 to 34.42 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.500	-14.40	24.90	0.58
Span # 2	2	11.500	-15.00	24.90	0.60
Span # 3	3	11.500	-15.00	24.90	0.60
+1.40D					
Span # 1	1	11.500	-6.37	24.90	0.26
Span # 2	2	11.500	-6.63	24.90	0.27
Span # 3	3	11.500	-6.63	24.90	0.27
+1.20D+1.60L					
Span # 1	1	11.500	-14.40	24.90	0.58
Span # 2	2	11.500	-15.00	24.90	0.60
Span # 3	3	11.500	-15.00	24.90	0.60
+1.20D+0.50L					
Span # 1	1	11.500	-8.25	24.90	0.33
Span # 2	2	11.500	-8.60	24.90	0.35
Span # 3	3	11.500	-8.60	24.90	0.35
+1.20D					
Span # 1	1	11.500	-5.46	24.90	0.22
Span # 2	2	11.500	-5.69	24.90	0.23
Span # 3	3	11.500	-5.69	24.90	0.23
+1.384D+0.50L					
Span # 1	1	11.500	-9.09	24.90	0.37
Span # 2	2	11.500	-9.47	24.90	0.38
Span # 3	3	11.500	-9.47	24.90	0.38
+0.90D					
Span # 1	1	11.500	-4.10	24.90	0.16
Span # 2	2	11.500	-4.27	24.90	0.17
Span # 3	3	11.500	-4.27	24.90	0.17
+0.7160D					
Span # 1	1	11.500	-3.26	24.90	0.13
Span # 2	2	11.500	-3.39	24.90	0.14
Span # 3	3	11.500	-3.39	24.90	0.14

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0129	5.290	+D+L	-0.0002	11.730
+D+L	2	0.0010	5.750	+D+L	-0.0008	1.150
+D+L	3	0.0129	6.210		0.0000	1.150

MYERS ENGINEERING

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Concrete Beam

File: 80xx SE 20th ST.ec6

Lic. # : KW-06008232
MYERS ENGINEERING

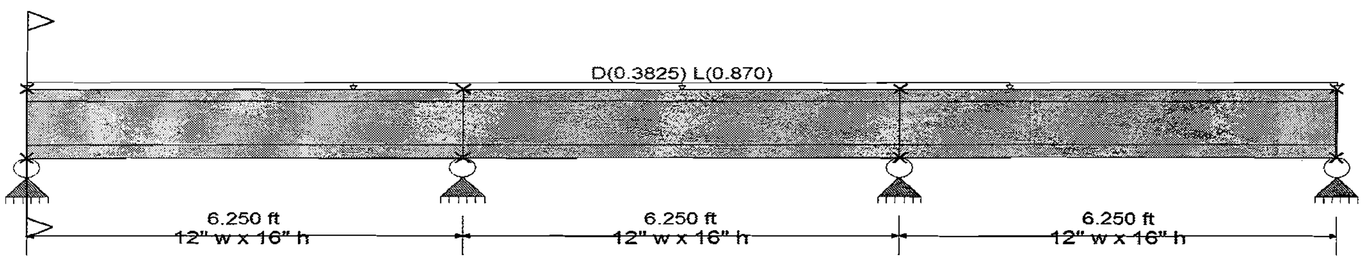
DESCRIPTION: E. Grade Beam supporting Den Wall

CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} * 7.50$	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2



Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 6.250 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 6.250 ft in this span
Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 6.250 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 6.250 ft in this span
Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 6.250 ft in this span — 2-#4 at 3.0 in from Bottom, from 0.0 to 6.250 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.3825, L = 0.870
Uniform Load on ALL spans : D = 0.3825, L = 0.870 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.327** : 1
Section used for this span	**Typical Section**
Mu : Applied	-8.137 k-ft
Mn * Phi : Allowable	24.904 k-ft
Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 3

Maximum Deflection			
Max Downward Transient Deflection	0.001 in	Ratio =	60938 >=720.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <720.0
Max Downward Total Deflection	0.002 in	Ratio =	36668 >=480
Max Upward Total Deflection	0.000 in	Ratio =	0 <480.0

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	3.615	9.940	9.940	3.615

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Concrete Beam

DESCRIPTION: E. Grade Beam supporting Den Wall

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	0.864	2.375	2.375	0.864
D Only	1.440	3.959	3.959	1.440
+D+L	3.615	9.940	9.940	3.615
+D+0.750L	3.071	8.445	8.445	3.071
+0.60D	0.864	2.375	2.375	0.864
L Only	2.175	5.981	5.981	2.175

Shear Stirrup Requirements

Between 0.00 to 5.33 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 5.38 to 6.50 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 6.54 to 12.21 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 12.25 to 13.38 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 13.42 to 18.71 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	6.250	-7.81	24.90	0.31
Span # 2	2	6.250	-8.14	24.90	0.33
Span # 3	3	6.250	-8.14	24.90	0.33
+1.40D					
Span # 1	1	6.250	-3.02	24.90	0.12
Span # 2	2	6.250	-3.15	24.90	0.13
Span # 3	3	6.250	-3.15	24.90	0.13
+1.20D+1.60L					
Span # 1	1	6.250	-7.81	24.90	0.31
Span # 2	2	6.250	-8.14	24.90	0.33
Span # 3	3	6.250	-8.14	24.90	0.33
+1.20D+0.50L					
Span # 1	1	6.250	-4.22	24.90	0.17
Span # 2	2	6.250	-4.40	24.90	0.18
Span # 3	3	6.250	-4.40	24.90	0.18
+1.20D					
Span # 1	1	6.250	-2.59	24.90	0.10
Span # 2	2	6.250	-2.70	24.90	0.11
Span # 3	3	6.250	-2.70	24.90	0.11
+1.384D+0.50L					
Span # 1	1	6.250	-4.62	24.90	0.19
Span # 2	2	6.250	-4.81	24.90	0.19
Span # 3	3	6.250	-4.81	24.90	0.19
+0.90D					
Span # 1	1	6.250	-1.94	24.90	0.08
Span # 2	2	6.250	-2.02	24.90	0.08
Span # 3	3	6.250	-2.02	24.90	0.08
+0.7160D					
Span # 1	1	6.250	-1.55	24.90	0.06
Span # 2	2	6.250	-1.61	24.90	0.06
Span # 3	3	6.250	-1.61	24.90	0.06

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0020	2.875	+D+L	-0.0000	6.375
+D+L	2	0.0002	3.125	+D+L	-0.0001	5.625
+D+L	3	0.0020	3.375		0.0000	5.625

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Concrete Beam

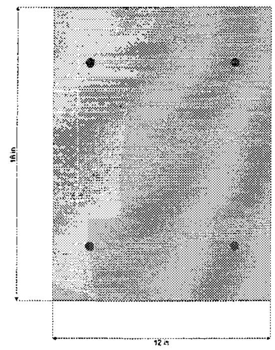
DESCRIPTION: F. Grade Beam at Garage Floor

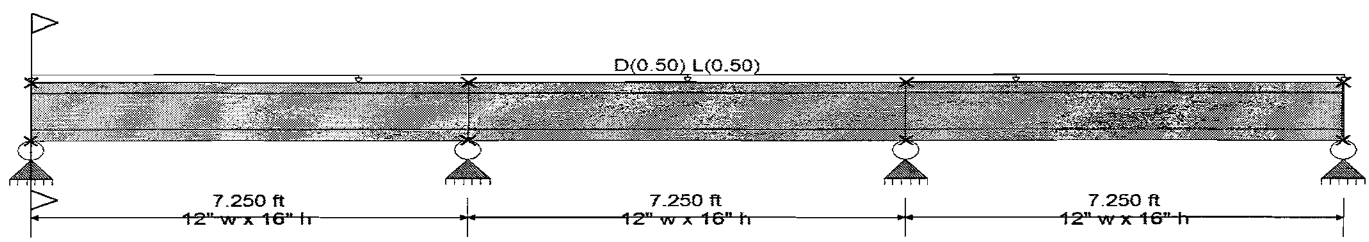
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

f'c =	2.50 ksi	φ Phi Values Flexure :	0.90
fr = $f'c^{1/2} * 7.50$ =	375.0 psi	Shear :	0.750
ψ Density =	145.0 pcf	β_1 =	0.850
λ LtWt Factor =	1.0		
Elastic Modulus =	3,122.0 ksi	Fy - Stirrups	40.0 ksi
fy - Main Rebar =	60.0 ksi	E - Stirrups =	29,000.0 ksi
E - Main Rebar =	29,000.0 ksi	Stirrup Bar Size #	3
		Number of Resisting Legs Per Stirrup =	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.250 ft in this span   2-#4 at 3.0 in from Bottom, from 0.0 to 7.250 ft in this span
Span #2 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.250 ft in this span   2-#4 at 3.0 in from Bottom, from 0.0 to 7.250 ft in this span
Span #3 Reinforcing....
2-#4 at 3.0 in from Top, from 0.0 to 7.250 ft in this span   2-#4 at 3.0 in from Bottom, from 0.0 to 7.250 ft in this span

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.050, L = 0.050
Uniform Load on ALL spans : D = 0.050, L = 0.050 ksf, Tributary Width = 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.344** : 1	Maximum Deflection		
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.001 in	Ratio = 67930 >=720
Mu : Applied	-8.578 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <720.0
Mn * Phi : Allowable	24.904 k-ft	Max Downward Total Deflection	0.003 in	Ratio = 28462 >=480
Location of maximum on span	0.000 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <480.0
Span # where maximum occurs	Span # 2			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 2	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77
Section 3	2- #4 @ d=3",2- #4 @ d=13",	24.90	24.90	4,096.00	472.77	472.77

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	3.461	9.517	9.517	3.461

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DESCRIPTION: F. Grade Beam at Garage Floor

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MINimum	1.206	3.318	3.318	1.206
D Only	2.011	5.529	5.529	2.011
+D+L	3.461	9.517	9.517	3.461
+D+0.750L	3.098	8.520	8.520	3.098
+0.60D	1.206	3.318	3.318	1.206
L Only	1.450	3.987	3.987	1.450

Shear Stirrup Requirements

Between 0.00 to 6.53 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 6.57 to 7.25 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 7.30 to 14.45 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in
Between 14.50 to 15.18 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.1, use #3 stirrups spaced at 6.000 in
Between 15.23 to 21.70 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use #3 stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	7.250	-8.24	24.90	0.33
Span # 2	2	7.250	-8.58	24.90	0.34
Span # 3	3	7.250	-8.58	24.90	0.34
+1.40D					
Span # 1	1	7.250	-4.90	24.90	0.20
Span # 2	2	7.250	-5.10	24.90	0.20
Span # 3	3	7.250	-5.10	24.90	0.20
+1.20D+1.60L					
Span # 1	1	7.250	-8.24	24.90	0.33
Span # 2	2	7.250	-8.58	24.90	0.34
Span # 3	3	7.250	-8.58	24.90	0.34
+1.20D+0.50L					
Span # 1	1	7.250	-5.46	24.90	0.22
Span # 2	2	7.250	-5.69	24.90	0.23
Span # 3	3	7.250	-5.69	24.90	0.23
+1.20D					
Span # 1	1	7.250	-4.20	24.90	0.17
Span # 2	2	7.250	-4.37	24.90	0.18
Span # 3	3	7.250	-4.37	24.90	0.18
+1.384D+0.50L					
Span # 1	1	7.250	-6.10	24.90	0.25
Span # 2	2	7.250	-6.36	24.90	0.26
Span # 3	3	7.250	-6.36	24.90	0.26
+0.90D					
Span # 1	1	7.250	-3.15	24.90	0.13
Span # 2	2	7.250	-3.28	24.90	0.13
Span # 3	3	7.250	-3.28	24.90	0.13
+0.7160D					
Span # 1	1	7.250	-2.51	24.90	0.10
Span # 2	2	7.250	-2.61	24.90	0.10
Span # 3	3	7.250	-2.61	24.90	0.10

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0031	3.335	+D+L	-0.0001	7.395
+D+L	2	0.0002	3.625	+D+L	-0.0002	0.725
+D+L	3	0.0031	3.915		0.0000	0.725

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x6 Built Up Post Properties

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

$h := (5.5) \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x6 Built Up Post Properties

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

$h := 5.5 \cdot in$

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

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

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

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

$C_p = 0.64$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

3-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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

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

$E' := 1200000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

2-2x4 Built Up Post Properties

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

$h := 3.5 \cdot in$

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

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

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

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

$C_p = 0.32$

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

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Maximum Load For 4x4 HF#2 Treated Post

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

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

$E' := 1235000 \cdot psi$

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

Axial Load Capacity

Slenderness Ratio (SL)

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

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

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

4x4 Treated Wood Post Properties

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

$h := 3.5 \cdot in$

$t := 3.5 \cdot in$

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

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

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

$C_p = 0.6$

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