REVIEWED FOR
COMPLIANCE

Conrad Tallariti

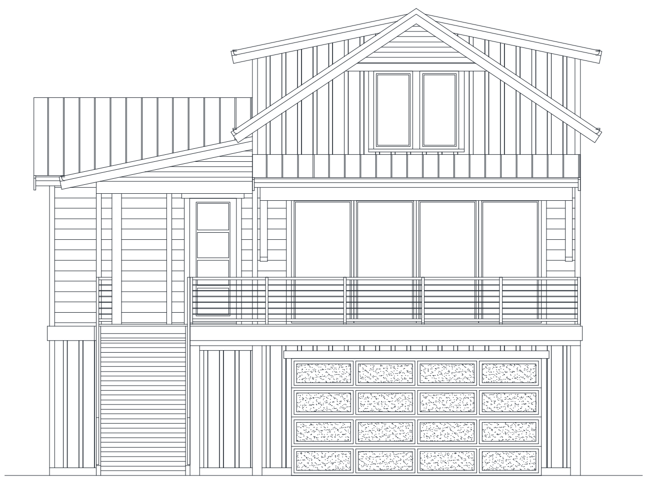
14039 Chimney Lane
Glacier, WA

Structural Calculations

Revision 1

DVDT Job No. - 24122

Drawings by JWR Design - # 24-075



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07/10/2024

DVDT

DVDT Engineering, Inc.
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Description:

JWR Design has asked for structural engineering calculations for a residential house.

Scope:

Perform the structural calculations necessary to build the residence according to code, and obtain a building permit.

Building Department:

The calculations and drawings will be submitted to Whatcom County Building Services.

References:

2021 IBC, ASCE 7-16, 2018 NDS

Loads:

Roof Load

R_1 = 2.5; R_2 = 8

R_{DL} = Roof Dead Load = 15 psf

D_{F1} = Roof Dead Load Factor = $\frac{1}{12}\text{sqrt}((R_1)^2 + (12)^2) = \frac{1}{12} \times \text{sqrt}((2.5)^2 + (12)^2)$ = 1.02

D_{F2} = Roof Dead Load Factor = $\frac{1}{12}\text{sqrt}((R_2)^2 + (12)^2) = \frac{1}{12} \times \text{sqrt}((8)^2 + (12)^2)$ = 1.20

R_{LL} = Roof Live Load = $20 \times 1.0 \times (1.2\text{-}0.05 \times R_1) = 20 \times 1.0 \times (1.2\text{-}0.05 \times 2.5)$ = 21.5 psf

Snow Load

P_g = Ground Snow Load = 127 psf

C_e = Exposure Factor = 1.0

C_t = Thermal Factor = 1.1

I = Importance Factor = 1.0

P_f = Flat-Roof Snow Load = $0.7\, C_e\, C_t\, I\, P_g = (0.7)(1)(1.1)(1)(81)$ = 62.4 psf

R_{SL} = Design Roof Snow Load = 127 psf

Wall Load

W_{DL} = Wall Dead Load = 12 psf

Floor Load

F_{DL} = Floor Dead Load = 13 psf; F_{DLD} = Deck Dead Load = 9 psf

F_{LL} = Floor Live Load = 40 psf; F_{LLD} = Deck Live Load = 60 psf

F_{LDH} = Deck Live Load = 120 psf

Seismic Load

Equivalent Lateral Force Procedure, Wood shear wall system, Seismic Design Cat. D, Category II, Site Class D

F_a = 1.2; S_S = 0.938

F_V = 0; S_1 = 0.331

$S_{MS} = F_a\, S_S = (1.2)(0.938)$ = 1.13

$S_{M1} = F_V\, S_1 = (0)(0.331)$ = 0

$S_{DS} = \frac{2}{3} \times S_{MS} = \frac{2}{3} \times 1.13$ = 0.750

$S_{D1} = \frac{2}{3} \times S_{M1} = \frac{2}{3} \times 0$ = 0

R = 6.5; I_O = Occupancy Importance Factor = 1.0

Wind Load - Enclosed, Simple Diaphragm Building

V_{ult} = 3-second gust wind speed = 100 mph; Exposure B

K_{zt} = 1.0

Allowable Soil Load

σ_{All} = Allowable Soil Load = 2000 psf

Concrete

$f_c' = 2500$ psi

f_{y4} = 40,000 psi - Grade 40 rebar; f_{y6} = 60,000 psi - Grade 60 rebar

Steel Properties

F_y = Rolled Shapes = 50 ksi; F_{y46} = HSS Shapes = 46 ksi

E_S = 29,000 ksi

Building Information

h_r = Ridge Height = 29.93 ft

h_{eB} = Basement Plate Height = 8.72 ft

h_{eM} = Main Floor Plate Height = 18.86 ft

h_{eU} = Upper Floor Plate Height = 26.58 ft

h_{mr} = Mean Roof Height = $\frac{1}{2} \times (h_r + h_{eU}) = \frac{1}{2}(29.93 + 26.58) = 28.3$ ft

L_T = Transverse horizontal dimension = 34 ft; L_L = Longitudinal horizontal dimension = 36.5 ft

W = Building Weight

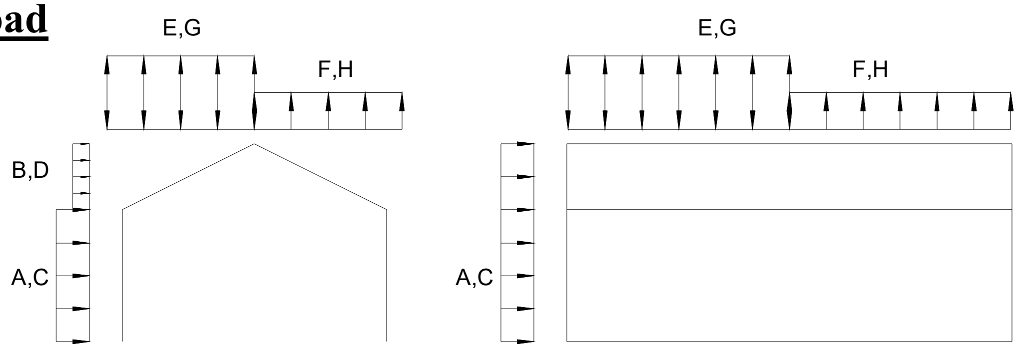
= $(634+965)(F_{DL}+5)+287\times(F_{DLD}+0.25\times F_{LDH})+(886+420+62)(R_{DL}+0.2\times R_{SL})+(115\times6.67+141\times9+141\times4)\times W_{DL}$ =

$(634+965)(13+5)+287\times(9+0.25\times120)+(886+420+62)(15+0.2\times127)+(115\times6.67+141\times9+141\times4)\times12$ = 126,443 lbs

C_S = Seismic Coefficient = $\frac{S_{DS}}{\frac{R}{I_O}} = \frac{0.75}{\frac{6.5}{1}}$ = 0.115 lbs

V_S = Seismic base shear = $1.3\times C_S\times W\times0.7 = 1.3\times0.115\times126{,}443\times0.7$ = **13,276 lbs**

Wind Load



$A_1 = 17.8$; $B_1 = 12.2$; $C_1 = 14.2$; $D_1 = 9.8$; $E_1 = 1.4$; $F_1 = -10.8$; $G_1 = 0.5$; $H_1 = -9.3$; $E_{OH1} = -6.3$; $G_{OH1} = -7.2$

$A_2 = 17.8$; $B_2 = 12.2$; $C_2 = 14.2$; $D_2 = 9.8$; $E_2 = 6.9$; $F_2 = -5.3$; $G_2 = 5.9$; $H_2 = -3.8$; $E_{OH2} = -6.3$; $G_{OH2} = -7.2$

$a_1 = 0.4\times h_{mr} = 0.4\times28.3$ = 11.3 ft

$a_2 = 0.1\times\min(L_T; L_L) = 0.1\times\min(34; 36.5)$ = 3.4 ft

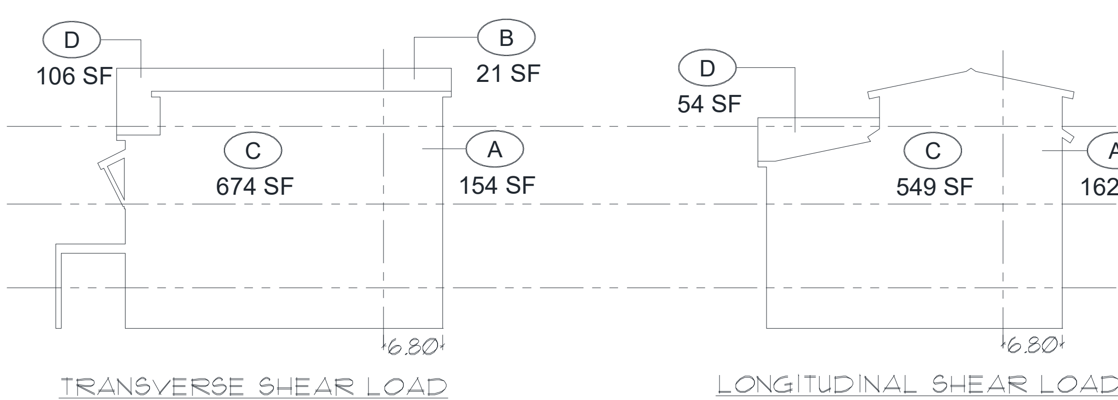
a_{min} = Minimum = 3 ft

λ = Height and Exposure Adjustment Factor (Figure 28.5-1) = 1.0

L_E = End zone length = $2\max(a_{min};\min(a_1;a_2)) = 2\times\max(3;\min(11.3;3.4))$ = 6.8 ft

$A_T = 154$; $B_T = 21$; $C_T = 674$; $D_T = 106$; $A_L = 162$; $B_L = 0$; $C_L = 549$; $D_L = 54$

$E_T = 90$; $F_T = 191$; $G_T = 373$; $H_T = 660$



Load Case 1

End Zone Pressures

$A_{LC1} = 0.6\times A_1\times\lambda\times K_{zt} = 0.6\times17.8\times1\times1 = 10.7$ psf
$B_{LC1} = 0.6\times B_1\times\lambda\times K_{zt} = 0.6\times12.2\times1\times1 = 7.32$ psf
$E_{LC1} = 0.6\times E_1\times\lambda\times K_{zt} = 0.6\times1.4\times1\times1 = 0.84$ psf
$F_{LC1} = 0.6\times F_1\times\lambda\times K_{zt} = 0.6\times(-10.8)\times1\times1 = -6.48$ psf

Interior Zone Pressures

$C_{LC1} = 0.6\times C_1\times\lambda\times K_{zt} = 0.6\times14.2\times1\times1 = 8.52$ psf
$D_{LC1} = 0.6\times D_1\times\lambda\times K_{zt} = 0.6\times9.8\times1\times1 = 5.88$ psf
$G_{LC1} = 0.6\times G_1\times\lambda\times K_{zt} = 0.6\times0.5\times1\times1 = 0.3$ psf
$H_{LC1} = 0.6\times H_1\times\lambda\times K_{zt} = 0.6\times(-9.3)\times1\times1 = -5.58$ psf

V_{wt1} = Trans Wind Shear = $A_{LC1}\times A_T+B_{LC1}\times B_T+C_{LC1}\times C_T+D_{LC1}\times D_T$
$= 10.7\times154+7.32\times21+8.52\times674+5.88\times106 =$ **8,164 lbs**

V_{ut1} = Transverse Wind Uplift = $E_{LC1}\times E_T+F_{LC1}\times F_T+G_{LC1}\times G_T+H_{LC1}\times H_T$
$= 0.84\times90+(-6.48)\times191+0.3\times373+(-5.58)\times660 =$ **-4,733 lbs**

V_{wl1} = Longitudinal Wind Shear = $A_{LC1}\times A_L+B_{LC1}\times B_L+C_{LC1}\times C_L+D_{LC1}\times D_L$
$= 10.7\times162+7.32\times0+8.52\times549+5.88\times54 =$ **6,725 lbs**

V_{ul1} = Longitudinal Wind Uplift = $E_{LC1}\times E_T+F_{LC1}\times F_T+G_{LC1}\times G_T+H_{LC1}\times H_T$
$= 0.84\times90+(-6.48)\times191+0.3\times373+(-5.58)\times660 =$ **-4,733 lbs**

Load Case 2

End Zone Pressures

$A_{LC2} = 0.6\times A_2\times\lambda\times K_{zt} = 0.6\times17.8\times1\times1 = 10.7$ psf
$B_{LC2} = 0.6\times B_2\times\lambda\times K_{zt} = 0.6\times12.2\times1\times1 = 7.32$ psf
$E_{LC2} = 0.6\times E_2\times\lambda\times K_{zt} = 0.6\times6.9\times1\times1 = 4.14$ psf
$F_{LC2} = 0.6\times F_2\times\lambda\times K_{zt} = 0.6\times(-5.3)\times1\times1 = -3.18$ psf

Interior Zone Pressures

$C_{LC2} = 0.6\times C_2\times\lambda\times K_{zt} = 0.6\times14.2\times1\times1 = 8.52$ psf
$D_{LC2} = 0.6\times D_2\times\lambda\times K_{zt} = 0.6\times9.8\times1\times1 = 5.88$ psf
$G_{LC2} = 0.6\times G_2\times\lambda\times K_{zt} = 0.6\times5.9\times1\times1 = 3.54$ psf
$H_{LC2} = 0.6\times H_2\times\lambda\times K_{zt} = 0.6\times(-3.8)\times1\times1 = -2.28$ psf

V_{wt2} = Trans Wind Shear = $A_{LC2}\times A_T+B_{LC2}\times B_T+C_{LC2}\times C_T+D_{LC2}\times D_T$
$= 10.7\times154+7.32\times21+8.52\times674+5.88\times106 =$ **8,164 lbs**

V_{ut2} = Transverse Wind Uplift = $E_{LC2}\times E_T+F_{LC2}\times F_T+G_{LC2}\times G_T+H_{LC2}\times H_T$
$= 4.14\times90+(-3.18)\times191+3.54\times373+(-2.28)\times660 =$ **-419 lbs**

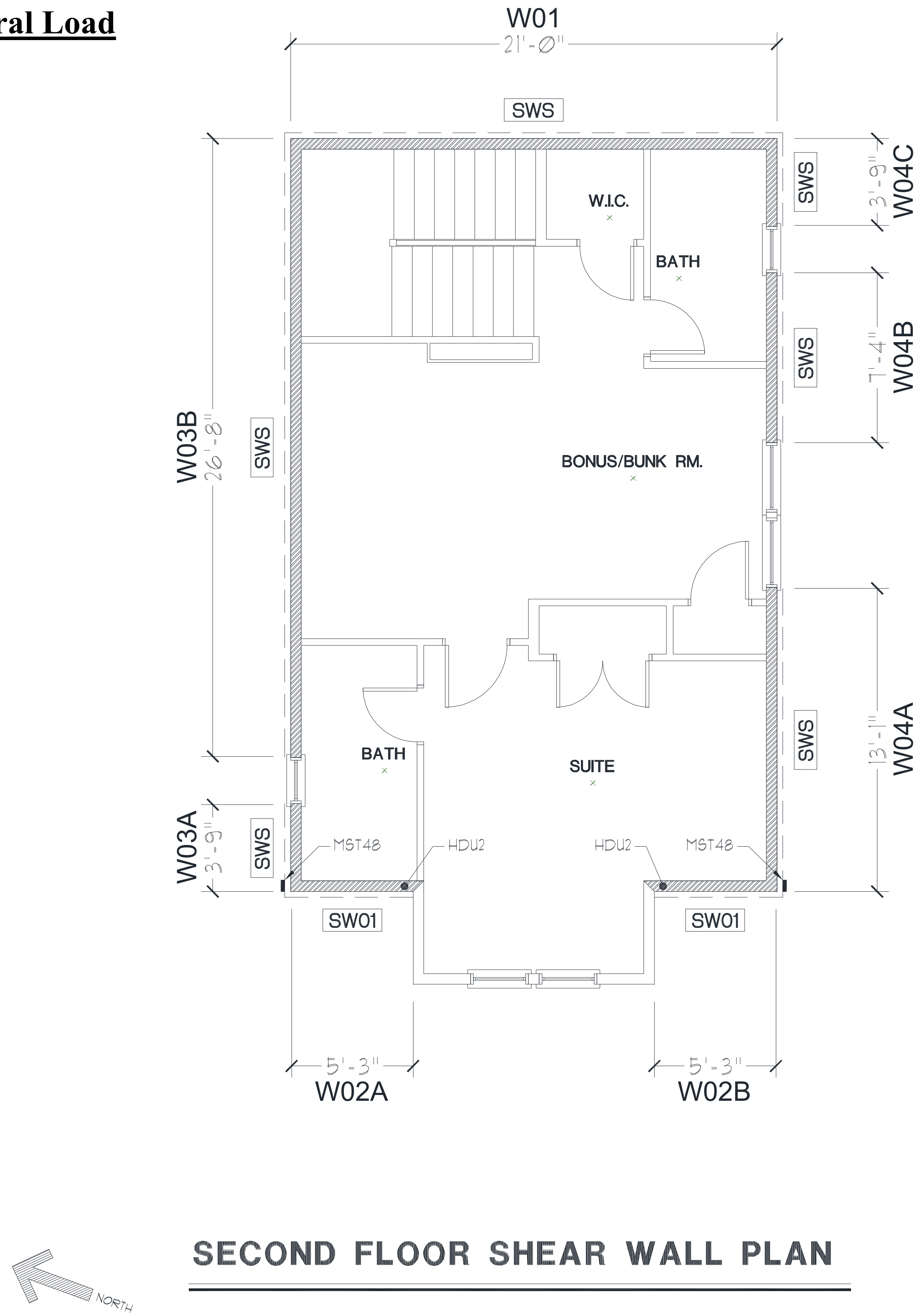
V_{wl2} = Longitudinal Wind Shear = $A_{LC2}\times A_L+B_{LC2}\times B_L+C_{LC2}\times C_L+D_{LC2}\times D_L$
$= 10.7\times162+7.32\times0+8.52\times549+5.88\times54 =$ **6,725 lbs**

V_{ul2} = Longitudinal Wind Uplift = $E_{LC2}\times E_T+F_{LC2}\times F_T+G_{LC2}\times G_T+H_{LC2}\times H_T$
$= 4.14\times90+(-3.18)\times191+3.54\times373+(-2.28)\times660 =$ **-419 lbs**

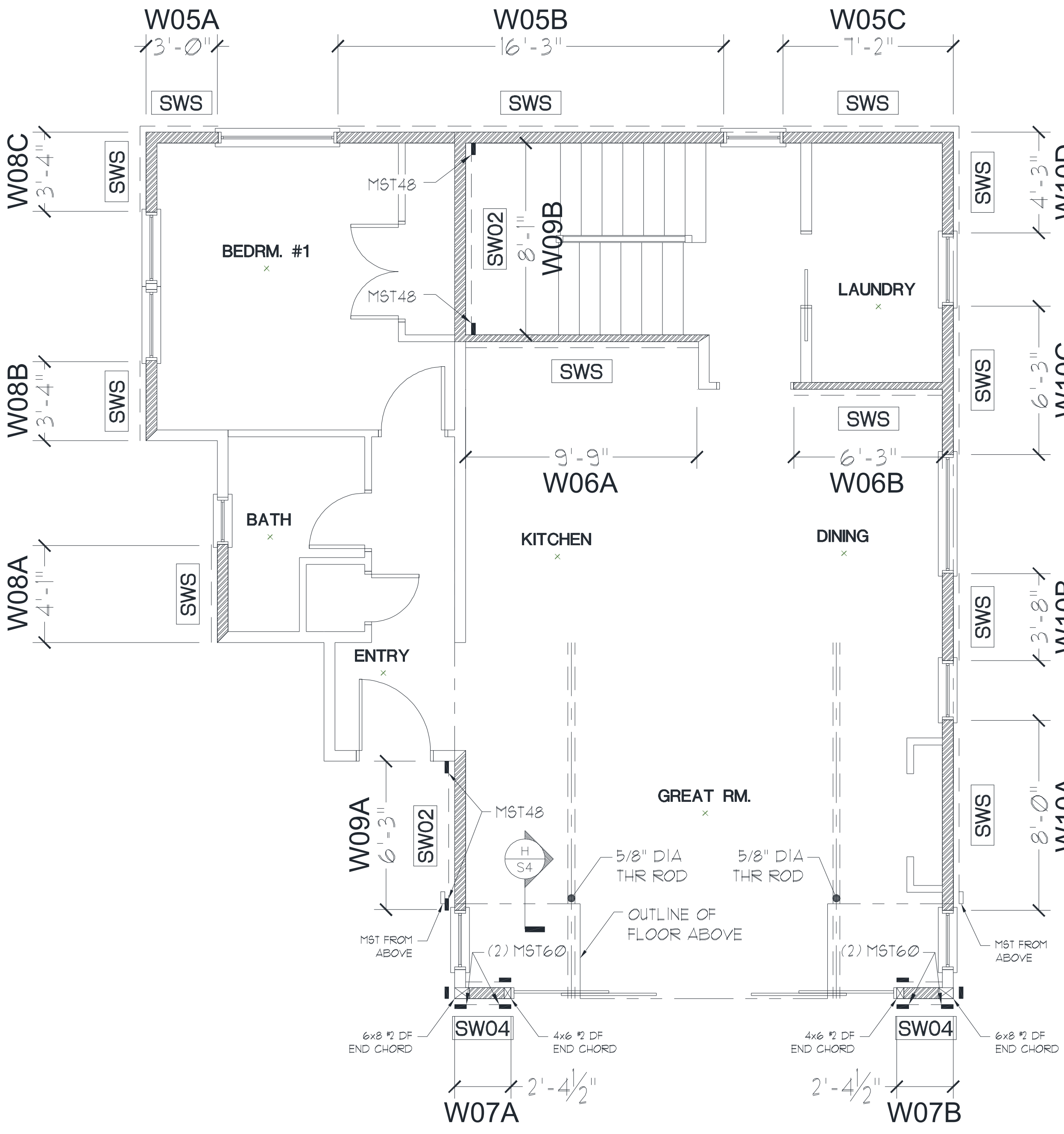
V_{wtmin} = Min Trans Wind = $0.6\times16\times(A_T+C_T)+0.6\times8\times(B_T+D_T) = 0.6\times16\times(154+674)+0.6\times8\times(21+106) =$ **8,558 lbs**
V_{wlmin} = Min Long Wind = $0.6\times16\times(A_L+C_L)+0.6\times8\times(B_L+D_L) = 0.6\times16\times(162+549)+0.6\times8\times(0+54) =$ **7,085 lbs**

Install Simpson H2.5 seismic ties or SDWC Rafter/Truss screws at each truss.

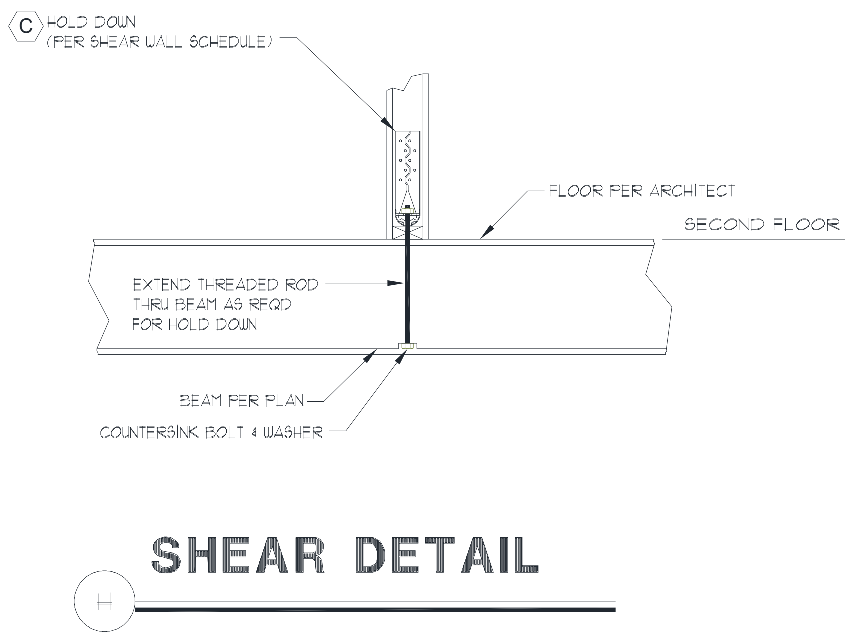
Lateral Load



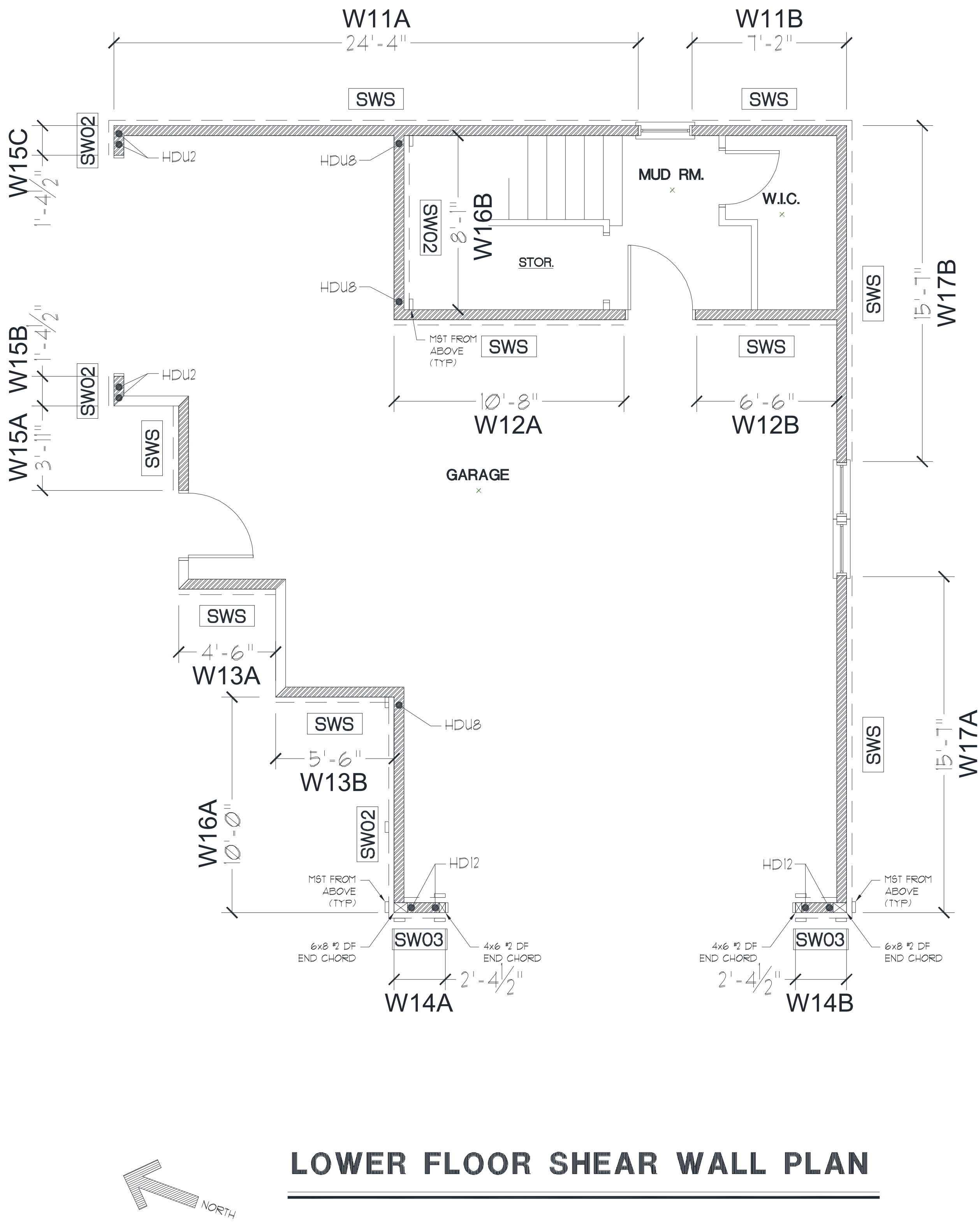
SECOND FLOOR SHEAR WALL PLAN



MAIN FLOOR SHEAR WALL PLAN



SHEAR DETAIL



LOWER FLOOR SHEAR WALL PLAN

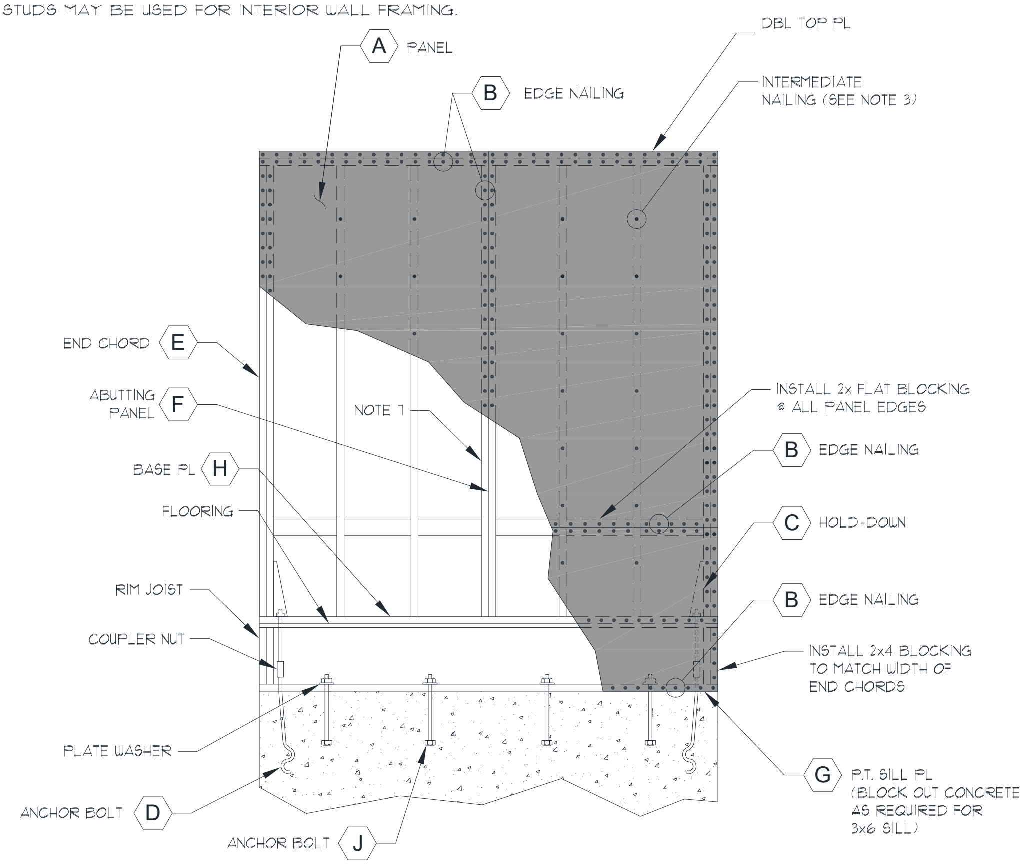
SHEAR WALL SCHEDULE

WALL DESC.	(A) PANELS	(B) EDGE NAIL	(C) HOLD DOWNS	(D) ANCHOR BOLTS	(E) END CHORDS 10	(F) ABUTTING PANELS 6 7	(G) SILL PL 8	(H) BASE PL CONN.	(J) SILL PL CONN.
SWS	7/16" OSB ONE SIDE	8d @ 6"	NONE	NONE	2x6	2x6	2x6	(3) 16d @ 16" o.c.	1/2" DIA. x 10" @ 60" o.c.
SW01	7/16" OSB ONE SIDE	8d @ 4"	SEE PLAN	SEE PLAN	DBL 2x6 DF	3x6	2x6	(3) 16d @ 12" o.c.	5/8" DIA. x 10" @ 16" o.c.
SW02	7/16" OSB ONE SIDE	8d @ 3"	SEE PLAN	SEE PLAN	DBL 2x6 DF	3x6	2x6	(3) 16d @ 8" o.c.	5/8" DIA. x 10" @ 12" o.c.
SW03	19/32" OSB BOTH SIDES	8d @ 2"	SEE PLAN	SEE PLAN	SEE PLAN	3x6	2x6	--	5/8" DIA. x 10" @ 12" o.c.
SW04	7/16" OSB ONE SIDE	8d @ 2"	SEE PLAN	SEE PLAN	SEE PLAN	3x6	2x6	(3) 16d @ 6" o.c.	5/8" DIA. x 10" @ 12" o.c.

DVDT No. 24122

NOTES:

1. ALL EXTERIOR WALLS SHALL HAVE MINIMUM ½" DIA. ANCHOR BOLTS SPACED AT 5'-0".
2. ALL TRUSSES AND RAFTERS SHALL BE INSTALLED WITH SIMPSON H2.5A HURRICANE TIES, OR SDWC RAFTER/TRUSS SCREWS.
3. ALL INTERMEDIATE FRAMING MEMBERS SHALL RECEIVE 8d NAILS @ 12" o.c.
4. ALL PANEL EDGES SHALL BE BLOCKED.
5. ALL HOLD DOWNS SHALL CONFORM TO SIMPSON STRONG-TIE.
6. ALTERNATE PANEL JOINTS WHEN PANELS ARE APPLIED TO BOTH SIDES OF THE WALL.
7. DBL 2X6 STUDS MAY BE USED IN LIEU OF 3x NOM STUDS FOR ABUTTING PANELS PROVIDING STUDS ARE NAILED W/ (2) 16d NAILS @ 6" o.c.
8. ALL SILL PLATES SHALL BE PT & INSTALLED USING 3 GA. 3"x3" WASHERS.
9. SHEAR WALL FRAMING SHALL BE CONSTRUCTED USING DF LUMBER.
10. MULTIPLE STUD END CHORDS SHALL BE NAILED W/ (2) 16d NAILS @ 8" O.C.
11. ALL NAILING TO PRESSURE TREATED LUMBER SHALL BE DONE USING HOT DIP GALVANIZED NAILS.
12. 2x4 STUDS MAY BE USED FOR INTERIOR WALL FRAMING.



SHEAR WALL LEGEND

Lateral Load Data

h_U = Upper floor shear wall height = 6.67 ft
h_M = First floor shear wall height = 9.1 ft
h_B = Basement shear wall height = 8.1 ft
C_D = 1.6;
Z_B = 860 lbs per 5/8" dia. bolt;
Z_N = 119 lbs per nail for 16d sinker nail
Z_{B2} = 590 lbs per 1/2" dia. bolt

Seismic Load Data

W_R = Roof weight = $886\times(R_{DL}+0.2\times R_{SL})+115\times3.335\times W_{DL} = 886\times(15+0.2\times127)+115\times3.335\times12$ = 40,397 lbs

W_{FU} = Upper Floor weight = $634\times(F_{DL}+5)+(420+62)(R_{DL}+0.2\times R_{SL})+(115\times3.335+141\times4.5)\times W_{DL} =$
$634\times(13+5)+(420+62)(15+0.2\times127)+(115\times3.335+141\times4.5)\times12$ = 43,101 lbs

W_{FM} = Main Floor weight = $965\times(F_{DL}+5)+ 287\times(F_{DLD}+0.25\times F_{LDH})+ (141\times4.5+141\times4)\times W_{DL}$
$= 965\times(13+5)+ 287\times(9+0.25\times120)+ (141\times4.5+141\times4)\times12$ = 42,945 lbs

W_T = Total Wt = $W_R+W_{FU}+W_{FM} = 40397+43101+42945$ = 126,443 lbs

S_{umVT} = Total Shear = $W_R\times h_{eU}+W_{FU}\times h_{eM}+W_{FM}\times h_{eB} = 40397\times26.58+43101\times18.86+42945\times8.72$ = 2,261,118 lbs

$V_R = W_R \times \frac{h_{eU}}{S_{umVT}} \times V_S = 40397 \times \frac{26.58}{2,261,118} \times 13276$ = 6,304 lbs

$V_{FU} = W_{FU} \times \frac{h_{eM}}{S_{umVT}} \times V_S = 43101 \times \frac{18.86}{2,261,118} \times 13276$ = 4,773 lbs

$V_{FM} = W_{FM} \times \frac{h_{eB}}{S_{umVT}} \times V_S = 42945 \times \frac{8.72}{2,261,118} \times 13276$ = 2,199 lbs

V_T = Total Shear = $V_R+V_{FU}+V_{FM} = 6,304+4,773+2,199$ = 13,276 lbs

w_{TR} = Roof Trans. shear = $\frac{V_R}{L_L} = \frac{6,304}{36.5}$ = **173 plf**

w_{TFU} = Upper Floor Trans. shear = $\frac{V_{FU}}{L_L} = \frac{4,773}{36.5}$ = **131 plf**

w_{TFM} = Main Floor Trans. shear = $\frac{V_{FM}}{L_L} = \frac{2,199}{36.5}$ = **60.2 plf**

w_{LR} = Roof Long. shear = $\frac{V_R}{21} = \frac{6,304}{21}$ = **300 plf**

w_{LFU} = Upper Floor Long. shear = $\frac{V_{FU}}{L_T} = \frac{4,773}{34}$ = **140 plf**

w_{LFM} = Main Floor Long. shear = $\frac{V_{FM}}{L_T} = \frac{2,199}{34}$ = **64.7 plf**

Wind Load Data - ASD

w_{TU} = Upper Transverse = $\frac{0.6\times16\times(28+104)+0.6\times8\times(B_T+D_T)}{L_L} = \frac{0.6\times16\times(28+104)+0.6\times8\times(21+106)}{36.5}$ = **51.4 lbs**

w_{LEU} = Upper Longitudinal end zone = $\frac{0.6\times16\times(35+80)+0.6\times8\times(0+0)}{21}$ = **52.6 lbs**

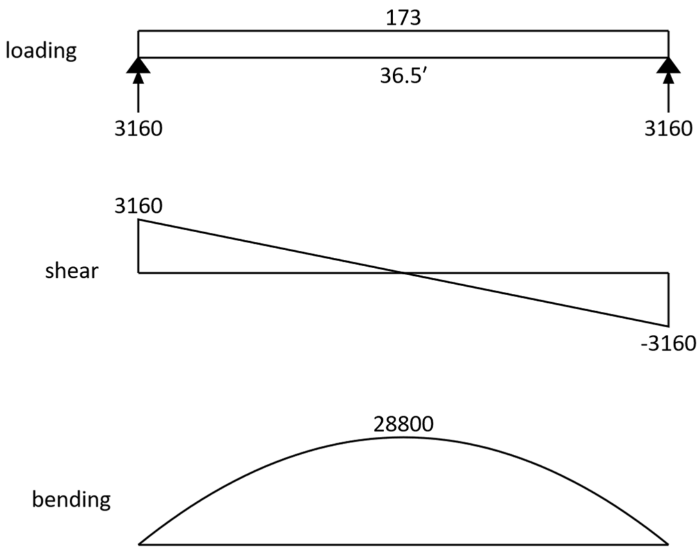
w_M = Main = $0.6\times16\times8.93+0.6\times8\times0$ = **85.7 lbs**

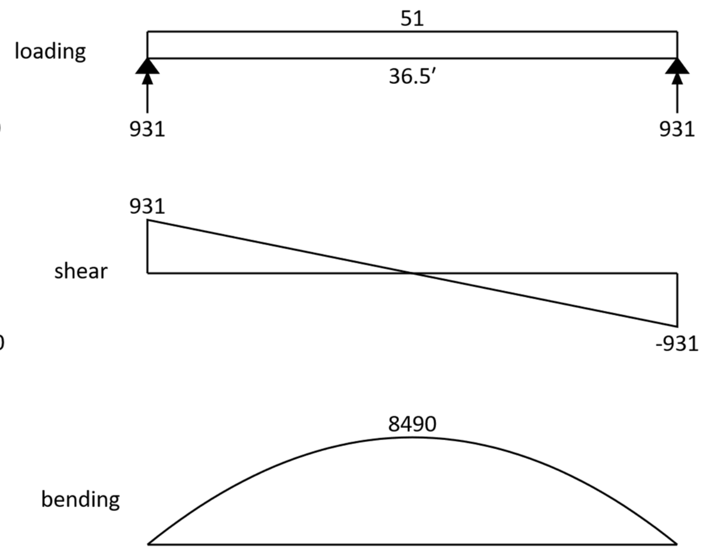
w_L = Lower = $0.6\times16\times9.65+0.6\times8\times0$ = **92.6 lbs**

Diaphragms

Transverse Upper Roof

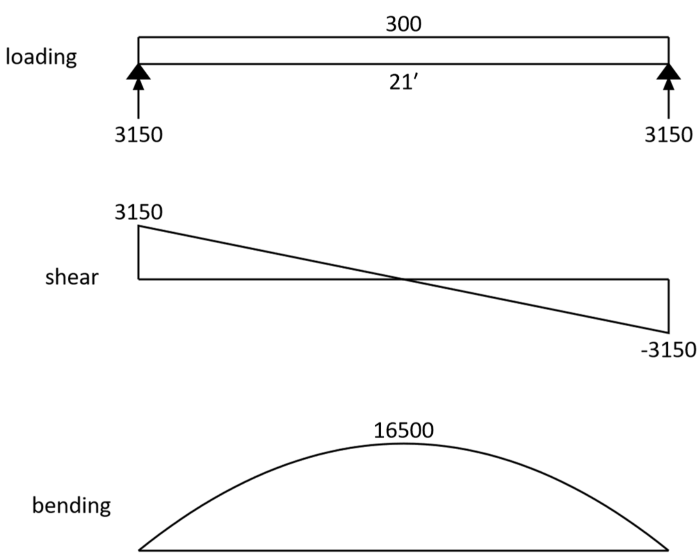
Seismic | **Wind**

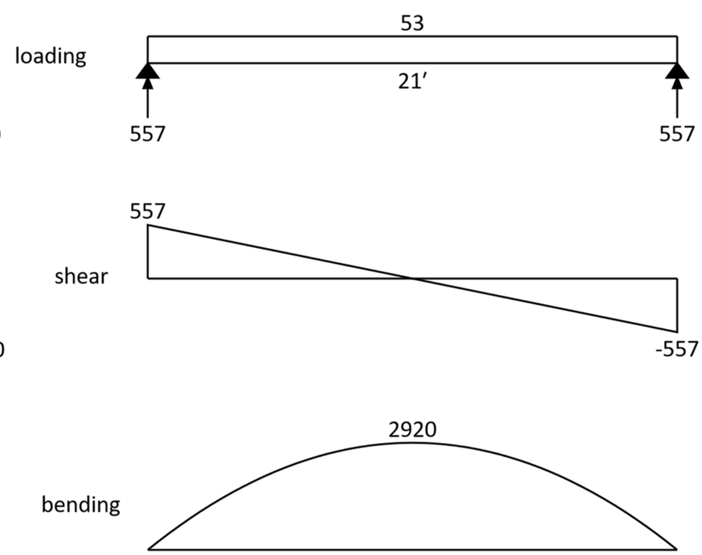




Longitudinal Upper Roof

Seismic | **Wind**





Job No.: 24122

DVDT

Date: 7/10/2024

Title: Conrad Tallariti

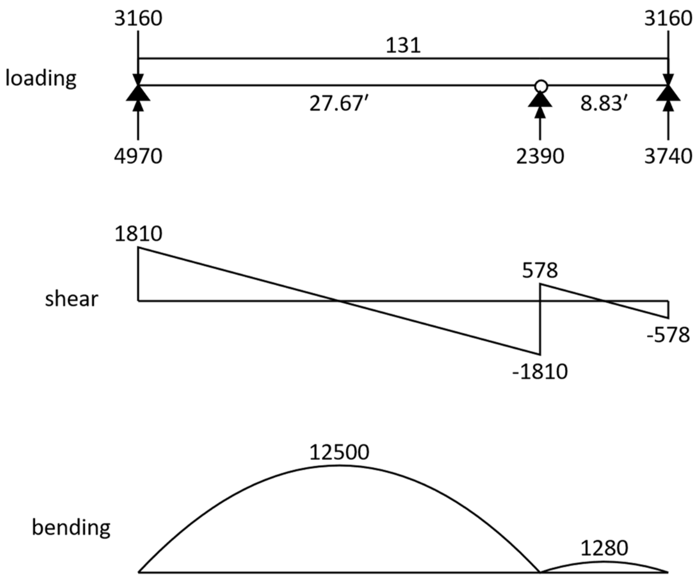
DVDT Engineering, Inc.

Loc.: 14039 Chimney Ln, Glacier

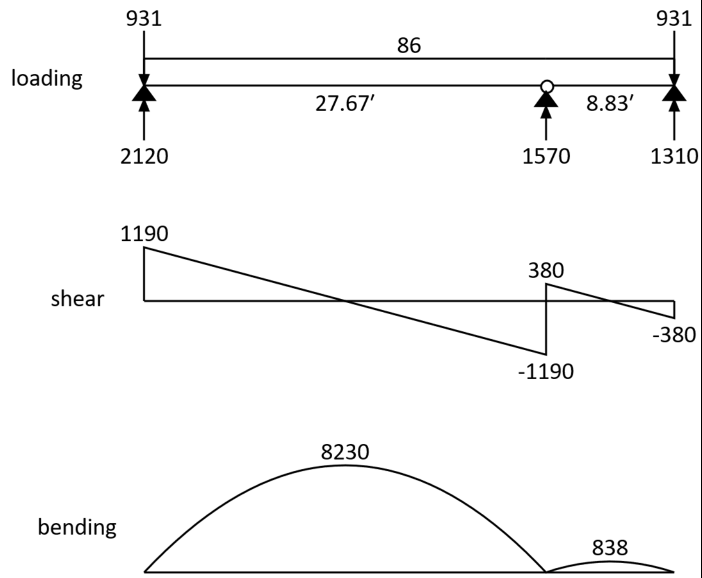
Structural Engineering

Transverse Second Floor

Seismic

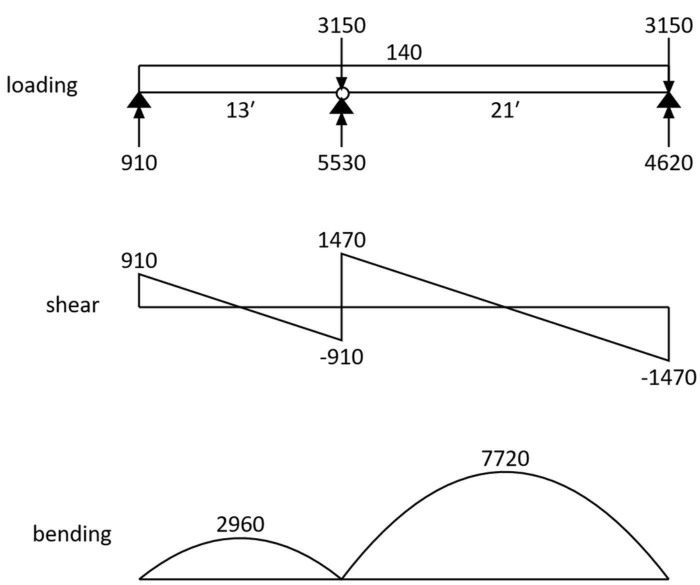


Wind

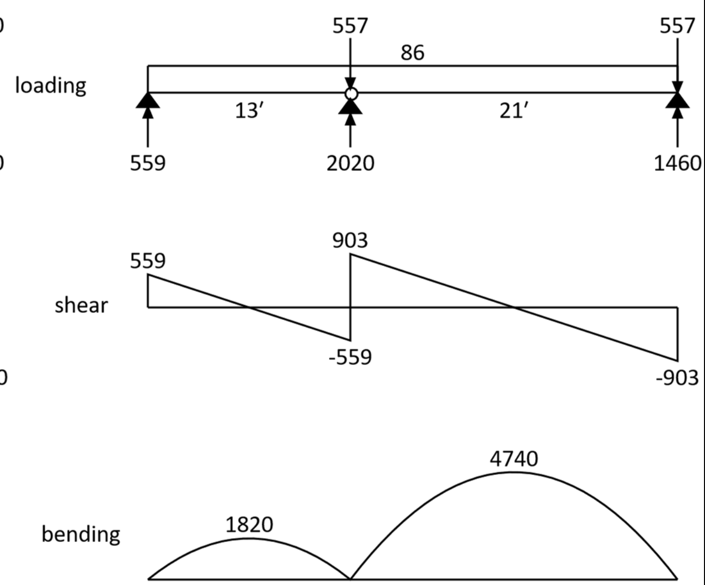


Longitudinal Second Floor

Seismic

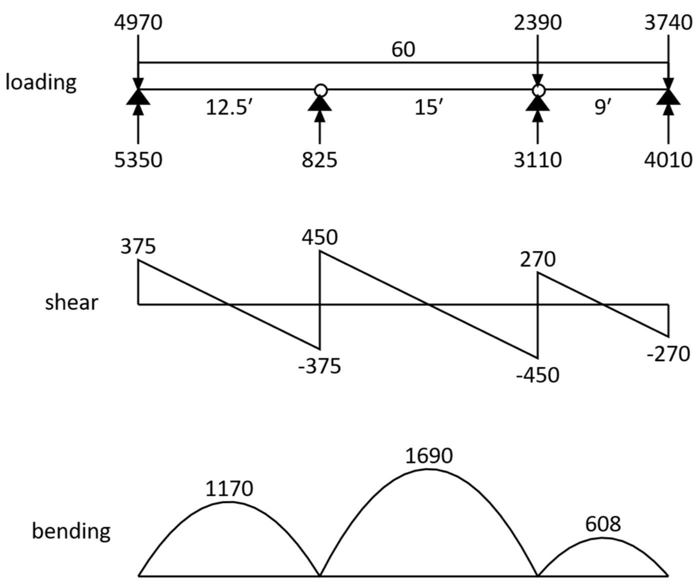


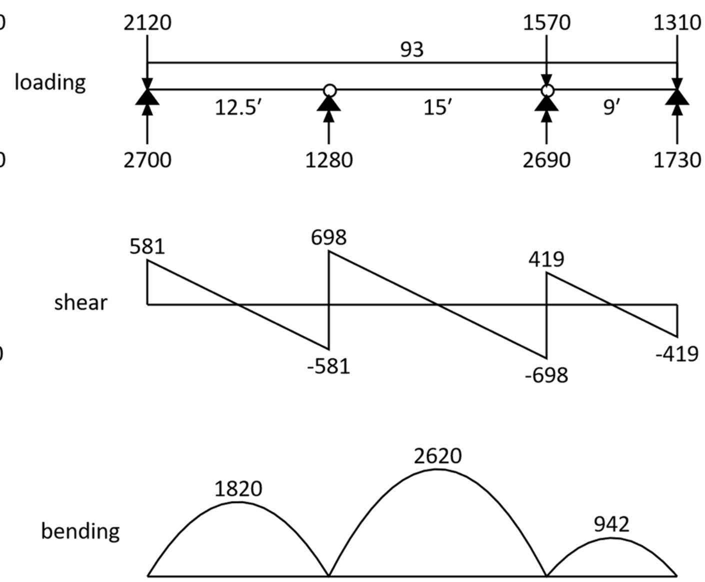
Wind



Transverse Main Floor

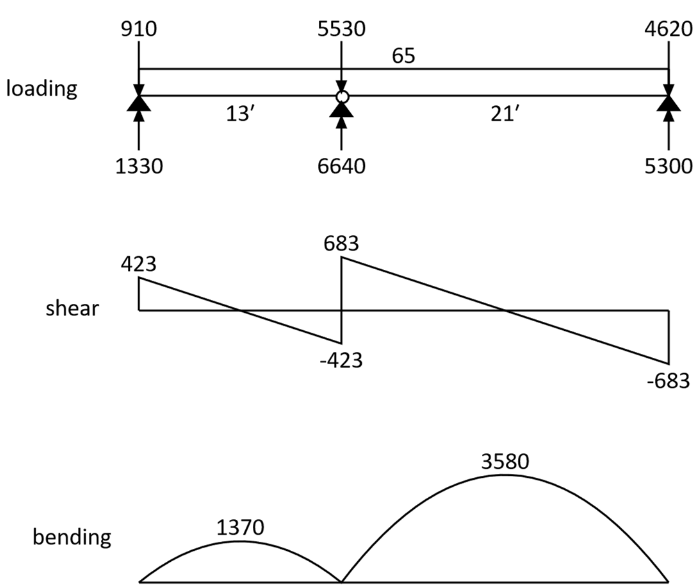
Seismic / Wind

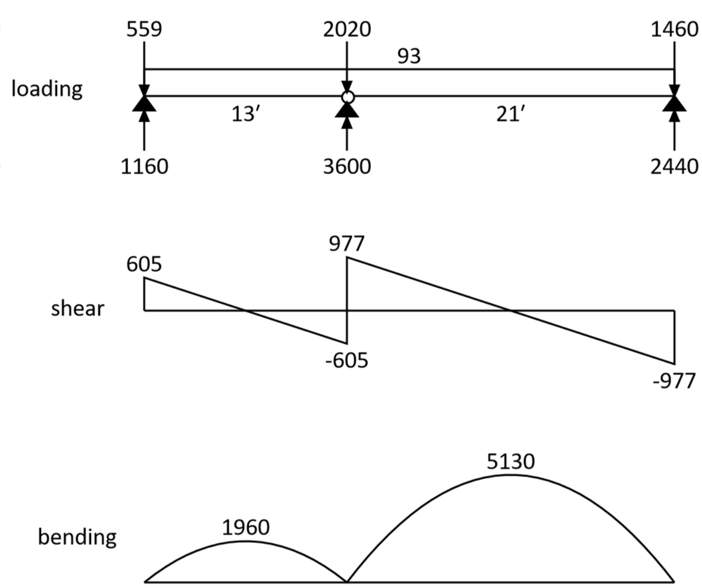




Longitudinal Main Floor

Seismic / Wind





W01 - East Wall - Second Floor

R_W = Wind Shear = 931 lbs
R_S = Seismic Shear = 3160 lbs
W01 = Length of Shear Wall = 21 ft
v_W = Wind Unit Shear = $\frac{R_W}{W01} = \frac{931}{21}$ = 44.3 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W01} = \frac{3160}{21}$ = 150 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\,h_U = (44.3)(6.67)$ = 296 lbs
T = C = Tension and Compression in the Chords = $v_S\,h_U = (150)(6.67)$ = 1,004 lbs

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W02 - West Walls - Second Floor

R_W = Wind Shear = 931 lbs
R_S = Seismic Shear = 3160 lbs
W02A = Length of Shear Wall = 5.25 ft
W02B = Length of Shear Wall = 5.25 ft
v_W = Wind Unit Shear = $\frac{R_W}{W02A+W02B} = \frac{931}{5.25+5.25}$ = 88.7 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W02A+W02B} = \frac{3160}{5.25+5.25}$ = 301 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\,h_U = (88.7)(6.67)$ = 591 lbs
T = C = Tension and Compression in the Chords = $v_S\,h_U = (301)(6.67)$ = 2,007 lbs

Shear Transfer Capacity

s_N = Triple Nail Spacing = 1.0 ft
s_B = Bolt Spacing = 1.33 ft

$Z_N' = 3 \times \left(\frac{1}{s_N}\right) Z_N C_D = 3 \times \left(\frac{1}{1}\right)(119)(1.6)$ = 571 lbs/ft

$Z_B' = \frac{1}{s_B} Z_B C_D = \frac{1}{1.33}(860)(1.6)$ = 1,035 lbs/ft

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{h_M}{W02A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{9.1}{5.25}\right) = 1.0$

$V_{WA} = \frac{1}{2} \times 1065 \times C_o = \frac{1}{2} \times 1065 \times 1.0$ = **532.5 plf**
$V_{SA} = \frac{1}{2} \times 760 \times C_o = \frac{1}{2} \times 760 \times 1.0$ = **380 plf**

Therefore: **7/16" OSB on one side of shear wall w/**
8d nails @ 4" o.c. at panel edges, and 12" in the field.
Use double stud end chords with MST48 or HDU2 hold downs.
Use (3) 16d nails @ 12" o.c. for wall to floor nailing.
Use 5/8" dia. shear bolts @ 16" o.c. for the P.T. sill to foundation connection.

Mark SW01 on the drawings.

W03 - North Walls - Second Floor

R_W = Wind Shear = 557 lbs
R_S = Seismic Shear = 3150 lbs

W03A = Length of Shear Wall = 3.75 ft
W03B = Length of Shear Wall = 26.67 ft

v_W = Wind Unit Shear = $\frac{R_W}{W03A+W03B} = \frac{557}{3.75+26.67}$ = 18.3 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W03A+W03B} = \frac{3150}{3.75+26.67}$ = 104 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_U$ = (18.3)(6.67) = 122 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_U$ = (104)(6.67) = 691 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

W04 - South Walls - Second Floor

R_W = Wind Shear = 557 lbs
R_S = Seismic Shear = 3150 lbs

W04A = Length of Shear Wall = 13.08 ft
W04B = Length of Shear Wall = 7.33 ft
W04C = Length of Shear Wall = 3.75 ft

v_W = Wind Unit Shear = $\frac{R_W}{W04A+W04B+W04C} = \frac{557}{13.08+7.33+3.75}$ = 23.1 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W04A+W04B+W04C} = \frac{3150}{13.08+7.33+3.75}$ = 130 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_U$ = (23.1)(6.67) = 154 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_U$ = (130)(6.67) = 867 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

W05 - East Walls - Main Floor

R_W = Wind Shear = 1310 lbs
R_S = Seismic Shear = 3740 lbs

W05A = Length of Shear Wall = 3 ft
W05B = Length of Shear Wall = 16.25 ft
W05C = Length of Shear Wall = 7.17 ft

v_W = Wind Unit Shear = $\frac{R_W}{W05A+W05B+W05C} = \frac{1310}{3+16.25+7.17}$ = 49.6 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W05A+W05B+W05C} = \frac{3740}{3+16.25+7.17}$ = 142 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_M$ = (49.6)(9.1) = 451 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M$ = (142)(9.1) = 1,288 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

W06 - Interior Stair Wall - Main Floor

R_W = Wind Shear = 1570 lbs
R_S = Seismic Shear = 2390 lbs

W06A = Length of Shear Wall = 9.75 ft
W06B = Length of Shear Wall = 6.25 ft

v_W = Wind Unit Shear = $\frac{R_W}{W06A+W06B} = \frac{1570}{9.75+6.25}$ = 98.1 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W06A+W06B} = \frac{2390}{9.75+6.25}$ = 149 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_M$ = $(98.1)(9.1)$ = 893 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M$ = $(149)(9.1)$ = 1,359 lbs

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W07 - West Walls - Main Floor

R_W = Wind Shear = 2120 lbs
R_S = Seismic Shear = 4970 lbs

W07A = Length of Shear Wall = 2.37 ft
W07B = Length of Shear Wall = 2.37 ft

v_W = Wind Unit Shear = $\frac{R_W}{W07A+W07B} = \frac{2120}{2.37+2.37}$ = 447 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W07A+W07B} = \frac{4970}{2.37+2.37}$ = 1,049 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_M$ = $(447)(9.1)$ = 4,070 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M$ = $(1{,}049)(9.1)$ = 9,542 lbs

Shear Transfer Capacity

s_N = Triple Nail Spacing = 0.5 ft
s_B = Bolt Spacing = 1.0 ft

$Z_N' = 3 \times \left(\frac{1}{s_N}\right) Z_N C_D = 3 \times \left(\frac{1}{0.5}\right)(119)(1.6)$ = 1,142 lbs/ft

$Z_B' = \frac{1}{s_B} Z_B C_D = \frac{1}{1}(860)(1.6)$ = 1,376 lbs/ft

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{8}{W07A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{8}{2.37}\right)$ = 0.828

$V_{WA} = 2 \times \frac{1}{2} \times 1790 \times C_o = 2 \times \frac{1}{2} \times 1790 \times 0.828$ = **1,482 plf**
$V_{SA} = 2 \times \frac{1}{2} \times 1280 \times C_o = 2 \times \frac{1}{2} \times 1280 \times 0.828$ = **1,060 plf**

Therefore: **7/16" OSB on one side of shear wall w/**
8d nails @ 2" o.c. at panel edges, and 12" in the field.
Use 4x6 #2 DF or 6x8 #2 DF end chords with (2) MST60 hold downs.
Use (3) 16d nails @ 6" o.c. for wall to floor nailing.
Use 5/8" dia. shear bolts @ 12" o.c. for the P.T. sill to foundation connection.

Mark SW04 on the drawings.

Shear Wall Deflection

A_B = Area of Boundary element = 2×1.5×5.5 = 16.5 in^2
b = Wall width = 2.17 ft
d_a = deflection due to anchorage details = 0.115 in for HDU5
E = 1600000 psi
v = Shear force = 4×v_S = (4)(190) = 760 lbs/ft
G_a = Apparent shear wall shear stiffness (Table 4.3A) = 39 kips/in
h = Wall height = 8 ft

Δ = Wall deflection = $\frac{8\times v\times h^3}{E\times A_B\times b}+\frac{v\times h}{2\times 1000\times G_a}+\frac{h}{b}\times d_a = \frac{8\times 760\times 8^3}{1600000\times 16.5\times 2.17}+\frac{760\times 8}{2\times 1000\times 39}+\frac{8}{2.17}\times 0.115$ = **0.556 in**

Δ_{allow} = Allowable Deflection = h×12×0.020 = 8×12×0.020 = **1.92 in** **OK**

Therefore the deflection in this wall is acceptable.

W08 - North Walls - Main Floor

R_W = Wind Shear = 559 lbs
R_S = Seismic Shear = 910 lbs
W08A = Length of Shear Wall = 4.08 ft
W08B = Length of Shear Wall = 3.33 ft
W08C = Length of Shear Wall = 3.33 ft

v_W = Wind Unit Shear = $\frac{R_W}{W08A+W08B+W08C} = \frac{559}{4.08+3.33+3.33}$ = 52.0 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W08A+W08B+W08C} = \frac{910}{4.08+3.33+3.33}$ = 84.7 lbs/ft

T = C = Tension and Compression in the Chords = $v_W\, h_M$ = (52)(9.1) = 473 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M$ = (84.7)(9.1) = 771 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

W09 - Intermediate Walls - Main Floor

R_W = Wind Shear = 2020 lbs
R_S = Seismic Shear = 5530 lbs

W09A = Length of Shear Wall = 6.25 ft
W09B = Length of Shear Wall = 8.08 ft

v_W = Wind Unit Shear = $\frac{R_W}{W09A+W09B} = \frac{2020}{6.25+8.08}$ = 141 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W09A+W09B} = \frac{5530}{6.25+8.08}$ = 386 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_M + 122 = (141)(9.1) + 122$ = 1,405 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M + 691 = (386)(9.1) + 691$ = 4,204 lbs

Shear Transfer Capacity

s_N = Triple Nail Spacing = 0.67 ft
s_B = Bolt Spacing = 1.0 ft

$Z_N' = 3 \times \left(\frac{1}{s_N}\right) Z_N C_D = 3 \times \left(\frac{1}{0.67}\right)(119)(1.6)$ = 853 lbs/ft

$Z_B' = \frac{1}{s_B} Z_B C_D = \frac{1}{1}(860)(1.6)$ = 1,376 lbs/ft

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{h_M}{W09A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{9.1}{6.25}\right) = 1.0$

$V_{WA} = \frac{1}{2} \times 1370 \times C_o = \frac{1}{2} \times 1370 \times 1$ = **685 plf**
$V_{SA} = \frac{1}{2} \times 980 \times C_o = \frac{1}{2} \times 980 \times 1$ = **490 plf**

Therefore: **7/16” OSB on one side of shear wall w/**
8d nails @ 3” o.c. at panel edges, and 12” in the field.
Use double stud end chords with MST48 hold downs.
Use (3) 16d nails @ 8” o.c. for wall to floor nailing.
Use 5/8” dia. shear bolts @ 12” o.c. for the P.T. sill to foundation connection.

Mark SW02 on the drawings.

W10 - South Walls - Main Floor

R_W = Wind Shear = 1460 lbs
R_S = Seismic Shear = 4620 lbs

W10A = Length of Shear Wall = 8 ft
W10B = Length of Shear Wall = 3.67 ft
W10C = Length of Shear Wall = 6.25 ft
W10D = Length of Shear Wall = 4.25 ft

v_W = Wind Unit Shear = $\frac{R_W}{W10A+W10B+W10C+W10D} = \frac{1460}{8+3.67+6.25+4.25}$ = 65.9 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W10A+W10B+W10C+W10D} = \frac{4620}{8+3.67+6.25+4.25}$ = 208 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_M = (65.9)(9.1)$ = 599 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M = (208)(9.1)$ = 1,896 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

W11 - East Walls - Lower Floor

R_W = Wind Shear = 1730 lbs
R_S = Seismic Shear = 4010 lbs

W11A = Length of Shear Wall = 24.33 ft
W11B = Length of Shear Wall = 7.17 ft

v_W = Wind Unit Shear = $\frac{R_W}{W11A+W11B} = \frac{1730}{24.33+7.17}$ = 54.9 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W11A+W11B} = \frac{4010}{24.33+7.17}$ = 127 lbs/ft

T = C = Tension and Compression in the Chords = $v_W\, h_B$ = (54.9)(8.1) = 445 lbs

T = C = Tension and Compression in the Chords = $v_S\, h_B$ = (127)(8.1) = 1,029 lbs

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W12 - Garage Rear Walls - Main Floor

R_W = Wind Shear = 2690 lbs
R_S = Seismic Shear = 3110 lbs

W12A = Length of Shear Wall = 10.67 ft
W12B = Length of Shear Wall = 6.5 ft

v_W = Wind Unit Shear = $\frac{R_W}{W12A+W12B} = \frac{2690}{10.67+6.5}$ = 157 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W12A+W12B} = \frac{3110}{10.67+6.5}$ = 181 lbs/ft

T = C = Tension and Compression in the Chords = $v_W\, h_B$ = (157)(8.1) = 1,272 lbs

T = C = Tension and Compression in the Chords = $v_S\, h_B$ = (181)(8.1) = 1,466 lbs

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W13 - Intermediate Walls - Lower Floor

R_W = Wind Shear = 1280 lbs
R_S = Seismic Shear = 825 lbs

W13A = Length of Shear Wall = 4.5 ft
W13B = Length of Shear Wall = 5.5 ft

v_W = Wind Unit Shear = $\frac{R_W}{W13A+W13B} = \frac{1280}{4.5+5.5}$ = 128 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W13A+W13B} = \frac{825}{4.5+5.5}$ = 82.5 lbs/ft

T = C = Tension and Compression in the Chords = $v_W\, h_B$ = (128)(8.1) = 1,037 lbs

T = C = Tension and Compression in the Chords = $v_S\, h_B$ = (82.5)(8.1) = 668 lbs

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W14 - West Walls - Main Floor

R_W = Wind Shear = 2700 lbs
R_S = Seismic Shear = 5350 lbs
W14A = Length of Shear Wall = 2.37 ft
W14B = Length of Shear Wall = 2.37 ft
v_W = Wind Unit Shear = $\frac{R_W}{W14A+W14B} = \frac{2700}{2.37+2.37}$ = 570 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W14A+W14B} = \frac{5350}{2.37+2.37}$ = 1,129 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_B$+4070 = (570)(8.1) +4070 = 8,687 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_B$+9542 = (1129)(8.1)+9542 = 18,687 lbs

$$DL = 0.6 \times \left(\frac{1}{2} \times (W14A + 16) \times \left(\left(\frac{1}{2} \times 15 + 1\right) \times R_{DL} \times D_{F1} + \frac{1}{2} \times (15+15) \times F_{DL} + \frac{1}{2} \times 8 \times F_{DLD} + (h_U + h_M + h_B) \times W_{DL}\right)\right) = 0.6 \times \left(\frac{1}{2}(2.37+16)\left(\left(\frac{1}{2} \times 15 + 1\right) \times 15 \times 1.02 + \frac{1}{2}(15+15) \times 13 + \frac{1}{2} \times 8 \times 9 + (6.67 + 9.1 + 8.1) \times 12\right)\right) = 3{,}568 \text{ lbs}$$

Shear Transfer Capacity

s_B = Bolt Spacing = 1.0 ft
$Z_B' = \frac{1}{s_B} Z_B C_D = \frac{1}{1}(860)(1.6)$ = 1,376 lbs/ft

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{7.5}{W14A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{7.5}{2.37}\right) = 0.854$

$V_{WA} = 2 \times \frac{1}{2} \times 2435 \times C_o = 2 \times \frac{1}{2} \times 2435 \times 0.854$ = **2,079 plf**
$V_{SA} = 2 \times \frac{1}{2} \times 1740 \times C_o = 2 \times \frac{1}{2} \times 1740 \times 0.854$ = **1,486 plf**

Therefore: 19/32" OSB on both sides of the shear wall w/
10d nails @ 2" o.c. at panel edges, and 12" in the field.
Use 4x6 #2 DF or 6x8 #2 DF end chords with HD12 hold downs (PAB8).
Use (3) 16d nails @ 6" o.c. for wall to floor nailing.
Use 5/8" dia. shear bolts @ 12" o.c. for the P.T. sill to foundation connection.

Mark SW03 on the drawings.

Shear Wall Deflection

A_B = Area of Boundary element = 3.5×5.5 = 19.25 in²
b = Wall width = *W14A* = 2.37 ft
d_a = deflection due to anchorage details = 0.171 in for HD12
E = 1600000 psi
v = Shear force = 4×v_S = 4×1129 = 4,516 lbs/ft
G_a = Apparent shear wall shear stiffness (Table 4.3A) = 48 kips/in
h = Wall height = 13.5 ft

Δ = Wall deflection = $\frac{8 \times v \times h^3}{E \times A_B \times b} + \frac{v \times h}{2 \times 1000 \times G_a} + \frac{h}{b} \times d_a = \frac{8 \times 4{,}516 \times 13.5^3}{1600000 \times 19.25 \times 2.37} + \frac{4{,}516 \times 13.5}{2 \times 1000 \times 48} + \frac{13.5}{2.37} \times 0.171$ = **2.83 in**

Δ_{allow} = Allowable Deflection = h×12×0.020 = 13.5×12×0.020 = **3.24 in** **OK**

Therefore the deflection in this wall is acceptable.

W15 - North Walls - Lower Floor

R_W = Wind Shear = 1160 lbs
R_S = Seismic Shear = 1330 lbs

W15A = Length of Shear Wall = 3.92 ft
W15B = Length of Shear Wall = 1.37 ft
W15C = Length of Shear Wall = 1.37 ft

v_W = Wind Unit Shear = $\frac{R_W}{W15A+W15B+W15C} = \frac{1160}{3.92+1.37+1.37}$ = 174 lbs/ft
v_S = Seismic Unit Shear = $\frac{R_S}{W15A+W15B+W15C} = \frac{1330}{3.92+1.37+1.37}$ = 200 lbs/ft
T = C = Tension and Compression in the Chords = $v_W\, h_B$+473 = (174)(8.1)+473 = 1,884 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_B$+771 = (200)(8.1)+771 = 2,389 lbs

W15A:

Standard 6" nailing of 7/16" OSB with no hold downs is acceptable (365, 260 plf allowable).

W15B & W15C:

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{h_B}{W15A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{8.1}{3.92}\right) = 0.992$

$V_{WA} = \frac{1}{2} \times 1370 \times C_o = \frac{1}{2} \times 1370 \times 0.992 =$ **<u>679 plf</u>**
$V_{SA} = \frac{1}{2} \times 980 \times C_o = \frac{1}{2} \times 980 \times 0.992 =$ **<u>486 plf</u>**

Therefore: **7/16" OSB on one side of shear wall w/**
8d nails @ 3" o.c. at panel edges, and 12" in the field.
Use double stud end chords with HDU2 hold downs (SSTB20).
Use (3) 16d nails @ 8" o.c. for wall to floor nailing.
Use 5/8" dia. shear bolts @ 12" o.c. for the P.T. sill to foundation connection.

Mark SW02 on the drawings.

Shear Wall Deflection

A_B = Area of Boundary element = 2×1.5×5.5 = 16.5 in²
b = Wall width = $W15B$ = 1.37 ft
d_a = deflection due to anchorage details = 0.088 in for HDU2
E = 1600000 psi
v = Shear force = 4×v_S = 4×227 = 908 lbs/ft
G_a = Apparent shear wall shear stiffness (Table 4.3A) = 25 kips/in
h = Wall height = h_B = 8.1 ft

Δ = Wall deflection = $\frac{8 \times v \times h^3}{E \times A_B \times b} + \frac{v \times h}{1000 \times G_a} + \frac{h}{b} \times d_a = \frac{8 \times 908 \times 8.1^3}{1600000 \times 16.5 \times 1.37} + \frac{908 \times 8.1}{1000 \times 25} + \frac{8.1}{1.37} \times 0.088$ = **<u>0.921 in</u>**
Δ_{allow} = Allowable Deflection = h×12×0.020 = 8.1×12×0.020 = **<u>1.94 in</u>** **<u>OK</u>**

Therefore the deflection in this wall is acceptable.

W16 - Intermediate Walls - Lower Floor

R_W = Wind Shear = 3600 lbs
R_S = Seismic Shear = 6640 lbs

W16A = Length of Shear Wall = 10 ft
W16B = Length of Shear Wall = 8.08 ft

v_W = Wind Unit Shear = $\frac{R_W}{W16A+W16B} = \frac{3600}{10+8.08}$ = 199 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W16A+W16B} = \frac{6640}{10+8.08}$ = 367 lbs/ft

T = C = Tension and Compression in the Chords = $v_W\, h_M$ +1405 = (199)(9.1) +1405 = 3,217 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_M$ +3512 = (367)(9.1) +3512 = 6,854 lbs

Allowable Shear Capacity

$C_o = \min\left(1.0; 1.25 - 0.125 \times \frac{h_M}{W02A}\right) = \min\left(1.0; 1.25 - 0.125 \times \frac{9.1}{5.25}\right) = 1.0$

$V_{WA} = \frac{1}{2} \times 1065 \times C_o = \frac{1}{2} \times 1065 \times 1.0$ = **532.5 plf**

$V_{SA} = \frac{1}{2} \times 760 \times C_o = \frac{1}{2} \times 760 \times 1.0$ = **380 plf**

Therefore: **7/16” OSB on one side of shear wall w/**
8d nails @ 4” o.c. at panel edges, and 12” in the field.
Use double stud end chords with HDU8 hold downs (SSTB28)
Use (3) 16d nails @ 12” o.c. for wall to floor nailing.
Use 5/8” dia. shear bolts @ 16” o.c. for the P.T. sill to foundation connection.

Mark SW01 on the drawings.

W17 - South Walls - Lower Floor

R_W = Wind Shear = 2440 lbs
R_S = Seismic Shear = 5300 lbs

W17A = Length of Shear Wall = 15.58 ft
W17B = Length of Shear Wall = 15.58 ft

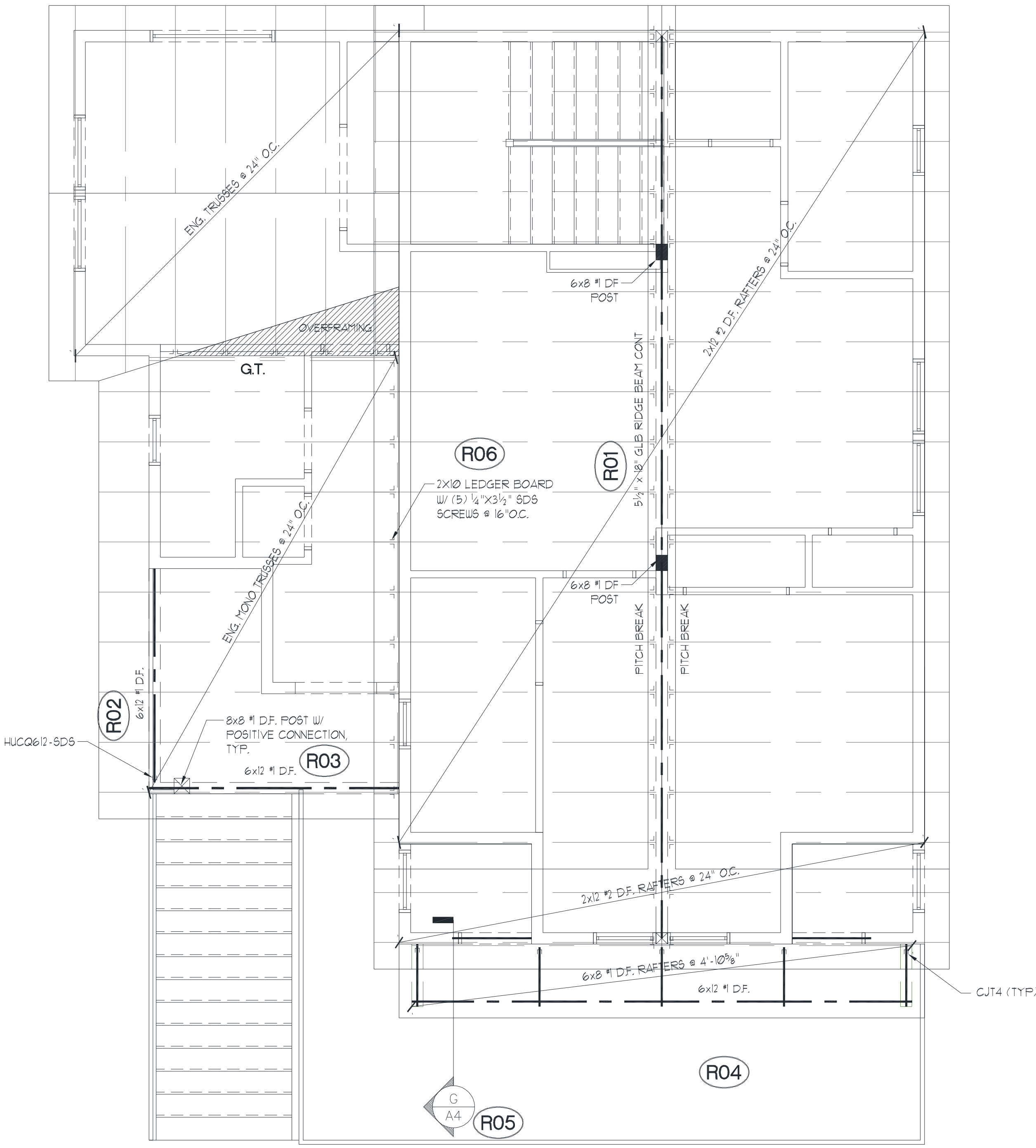
v_W = Wind Unit Shear = $\frac{R_W}{W17A+W17B} = \frac{2440}{15.58+15.58}$ = 78.3 lbs/ft

v_S = Seismic Unit Shear = $\frac{R_S}{W17A+W17B} = \frac{5300}{15.58+15.58}$ = 170 lbs/ft

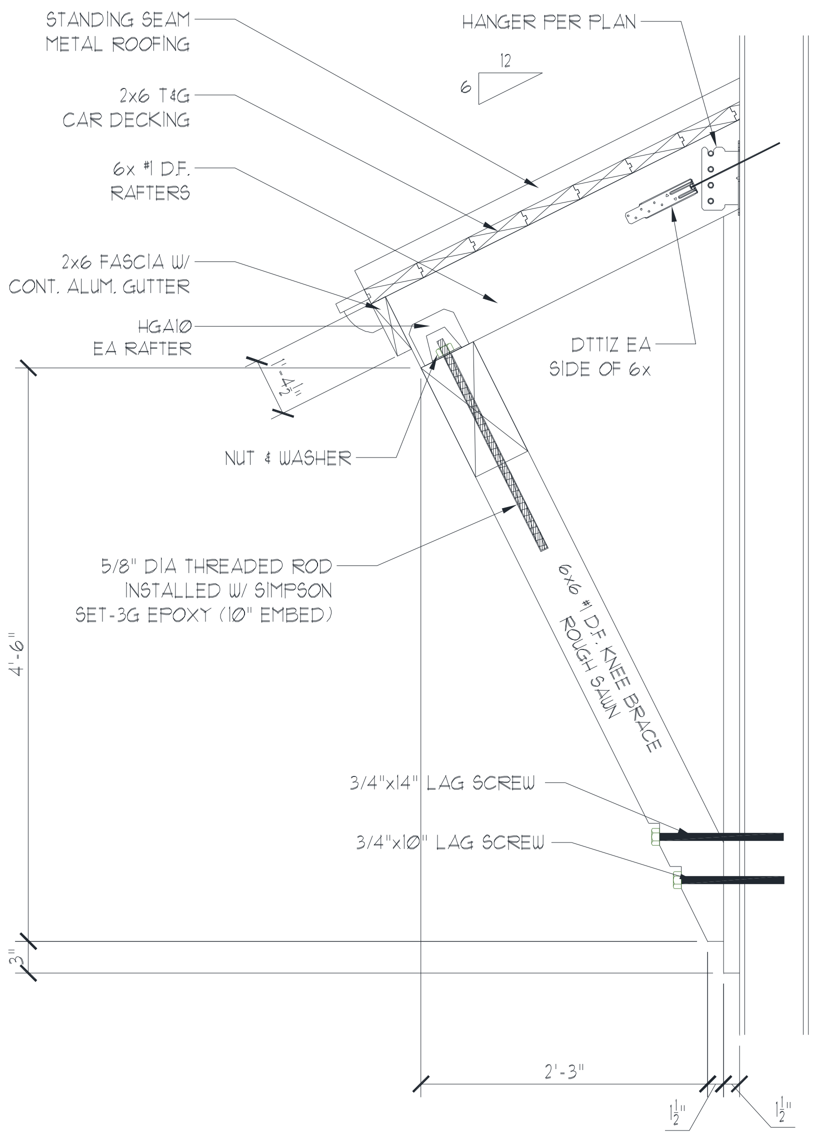
T = C = Tension and Compression in the Chords = $v_W\, h_B$ = (78.3)(8.1) = 634 lbs
T = C = Tension and Compression in the Chords = $v_S\, h_B$ = (170)(8.1) = 1,378 lbs

Standard 6” nailing of 7/16” OSB with no hold downs is acceptable (365, 260 plf allowable).

Vertical Load

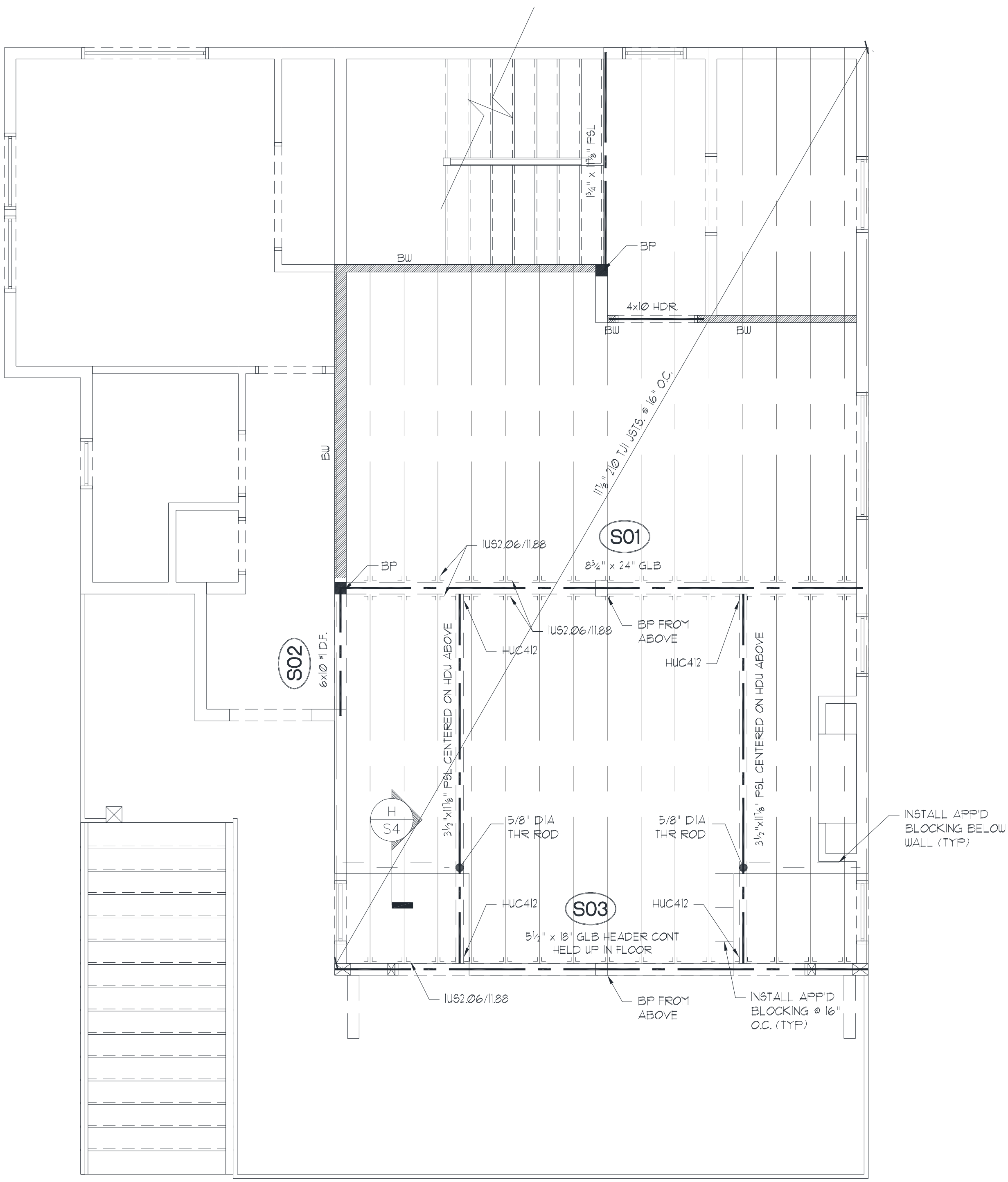


ROOF FRAMING PLAN

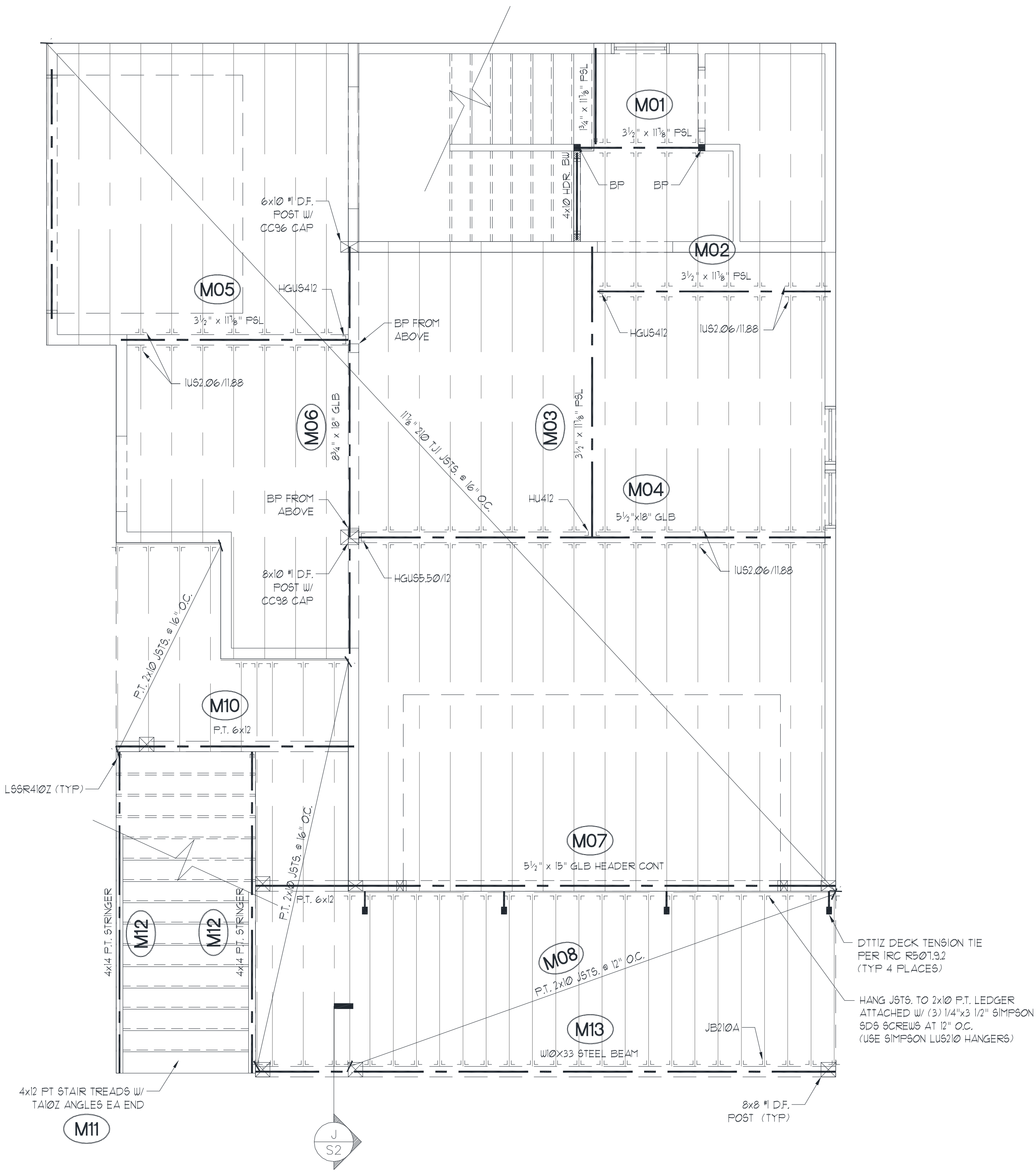


KNEE BRACE DETAIL

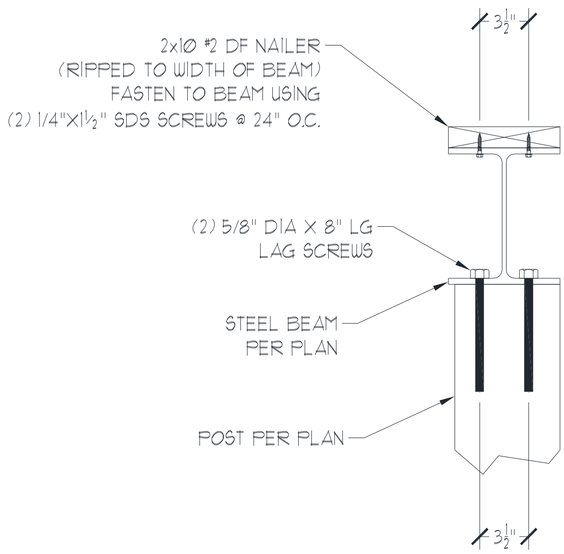
G



SECOND FLOOR FRAMING PLAN



MAIN FLOOR FRAMING PLAN



STEEL POST CONNECTION

J

R01 - 5 1/2” x 18” GLB Roof Ridge Beam - Continuous

b = Width = 5.5 in

d = Depth = 18 in

l_b = Bearing Length (assumed) = 7.5 in = 0.625 ft

w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 18$ = 24.1 lb/ft

L_{span} = Length of span = 15 ft

L_{ft} = Floor Tributary area = 0 ft

L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 21$ = 10.5 ft

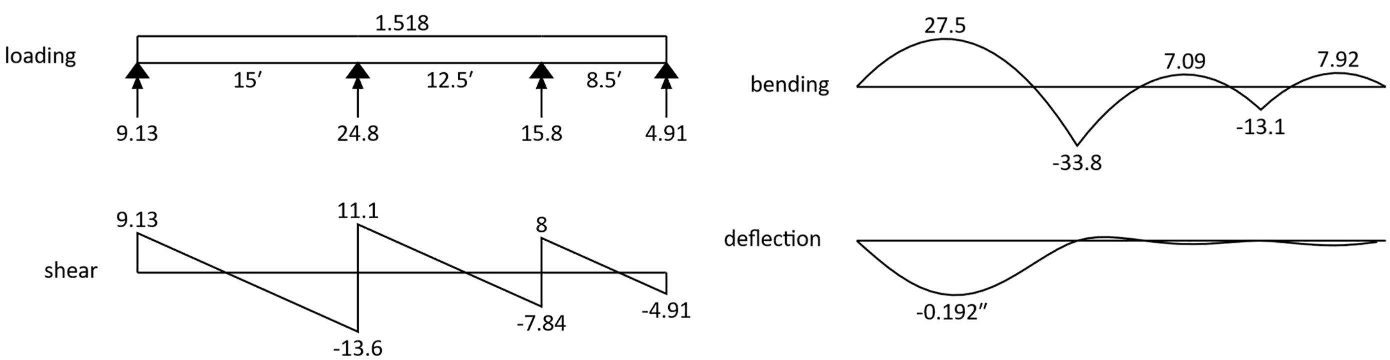
h_w = Wall Height = 0 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 24.1 + 0 \times 13 + 10.5 \times 15 \times 1.02 + 0 \times 12$ = 185 plf

W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL} = 0 \times 40 + 10.5 \times 127$ = 1,334 plf

W = Total Load = $W_D + W_L = 185 + 1{,}334$ = 1,518 plf

Forces



Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6} b\, d^2 = \left(\frac{1}{6}\right)(5.5)(18^2)$ = 297 in³

$I_x = \frac{1}{12} b\, d^3 = \left(\frac{1}{12}\right)(5.5)(18^3)$ = 2,673 in⁴

$A = b\, d = (5.5)(18)$ = 99 in²

E = 1800000 psi

C_D = 1.15

C_V = NDS 5.3.6 = $\min\left(1.0; \left(\frac{21}{L_{span}}\right)^{0.1} \left(\frac{12}{d}\right)^{0.1} \left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0; \left(\frac{21}{15}\right)^{0.1} \left(\frac{12}{18}\right)^{0.1} \left(\frac{5.125}{5.5}\right)^{0.1}\right)$ = 0.986

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi

$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 650 \times 5.5 \times 0.625 \times 12$ = **26,813 lbs**

F_v = Design shear stress = 265 psi

$V_{All} = F_v\, C_D \times A = (265)(1.15) \times 99$ = **30,170 lbs**

F_b = Design bending stress = 1850 psi

$M_{All} = F_b C_D C_V \left(\frac{S_x}{12}\right) = (1850)(1.15)(0.986)\left(\frac{297}{12}\right)$ = **51,918 ft·lbs**

Deflection

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{15 \times 12}{240}$ = 0.75 in **OK**

Therefore use a 5 1/2” x 18” GLB for the beam with 6x8 #1 DF center posts.

R02 - 6x12 #1 DF Porch Roof Beam

b = Width = 5.5 in
d = Depth = 11.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 11.5$ = 15.4 lb/ft
L_{span} = Length of span = 10 ft
L_{ft} = Floor Tributary area = 0 ft
L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 12 + 1 = 7$ ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 15.4+0×13 +7×15×1.02 +0×12 = 122 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = 0×40 +7×127 = 889 plf
W = Total Load = $W_D + W_L$ = 122 + 889 = 1,011 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(1{,}011)(10)$ = **5,057 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 1{,}011 \times 10^2$ = **12,643 ft·lbs**

Allowable Forces - #1 DF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(5.5)(11.5^2)$ = 121 in³
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(5.5)(11.5^3)$ = 697 in⁴
$A = b\ d = (5.5)(11.5)$ = 63.25 in²
E = 1600000 psi
C_D = Duration = 1.15
C_F = NDS Table 4D = 1.0
F_{cPerp} = Design compression stress perpendicular to grain = 625 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12$ = 625×5.5×0.125×12 = **5,156 lbs**
F_v = Design shear stress = 170 psi
$V_{All} = F_v\ C_D \times A$ = (170)(1.15)×63.25 = **12,365 lbs**
F_b = Design bending stress = 1350 psi
$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (1350)(1.15)(1)\left(\frac{121}{12}\right)$ = **15,684 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 889 \times \frac{1}{12}(10 \times 12)^4}{(384)(1600000)(697)}$ = 0.179 in

Allowable Live Load Deflection = $\frac{L_{span} \times 12}{360} = \frac{10 \times 12}{360}$ = 0.333 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 1{,}011 \times \frac{1}{12}(10 \times 12)^4}{(384)(1600000)(697)}$ = 0.204 in

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{10 \times 12}{240}$ = 0.5 in **OK**

Therefore use a 6x12 #1 DF for the beam with HUCQ612-SDS hanger.

R03 - 6x12 #1 DF Porch Roof Beam

b = Width = 5.5 in

d = Depth = 11.5 in

l_b = Bearing Length (assumed) = 7.5 in = 0.625 ft

w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d$ = $35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 11.5$ = 15.4 lb/ft

L_{span} = Length of span = 9 ft

L_C = Length of cantilever = 1.25 ft

L_{ft} = Floor Tributary area = 0 ft

L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 2 + 1 = 2$ ft

h_w = Wall Height = 0 ft

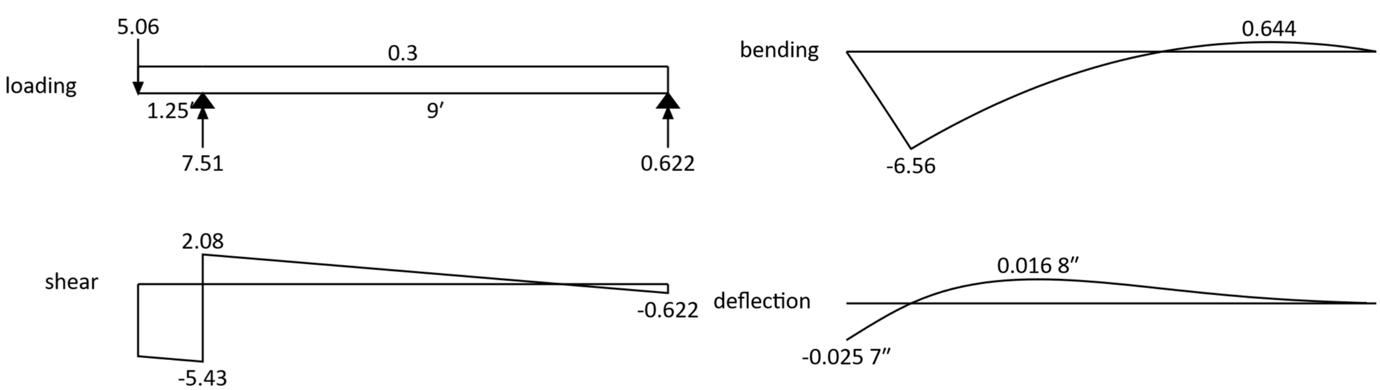
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = $15.4 + 0 \times 13 + 2 \times 15 \times 1.02 + 0 \times 12$ = 46.0 plf

W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = $0 \times 40 + 2 \times 127$ = 254 plf

W = Total Load = $W_D + W_L$ = $46.0 + 254$ = 300 plf

P = From R02 = 5,057 lbs

Forces



Allowable Forces - #1 DF

$S_x = \frac{1}{6} b\, d^2 = \left(\frac{1}{6}\right)(5.5)(11.5^2)$ = 121 in³

$I_x = \frac{1}{12} b\, d^3 = \left(\frac{1}{12}\right)(5.5)(11.5^3)$ = 697 in⁴

$A = b\, d = (5.5)(11.5)$ = 63.25 in²

E = 1600000 psi

C_D = Duration = 1.15

C_F = NDS Table 4D = 1.0

F_{cPerp} = Design compression stress perpendicular to grain = 625 psi

R_{All} **=** $F_{cPerp} \times b \times l_b \times 12$ **=** $625 \times 5.5 \times 0.625 \times 12$ = **<u>25,781 lbs</u>**

F_v = Design shear stress = 170 psi

V_{All} **=** $F_v\, C_D \times A$ **=** $(170)(1.15) \times 63.25$ = **<u>12,365 lbs</u>**

F_b = Design bending stress = 1350 psi

$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (1350)(1.15)(1)\left(\frac{121}{12}\right)$ = **<u>15,684 ft·lbs</u>**

Deflection

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{9 \times 12}{240}$ = 0.45 in **<u>OK</u>**

Therefore use a 6x12 #1 DF for the beam.

R04 - 6x12 #1 DF Knee Brace Roof Beam

b = Width = 5.5 in
d = Depth = 11.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 11.5$ = 15.4 lb/ft
L_{span} = Length of span = 19.5 ft
L_{ft} = Floor Tributary area = 0 ft
L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 2.25 + 0.5 = 1.625$ ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 15.4 + 0 \times 13 + 1.625 \times 15 \times 1.02 + 0 \times 12$ = 40.2 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL} = 0 \times 40 + 1.625 \times 127$ = 206 plf
W = Total Load = $W_D + W_L = 40.2 + 206$ = 247 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(247)(19.5)$ = **2,404 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 247 \times 19.5^2$ = **11,722 ft·lbs**

Allowable Forces - #1 DF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(5.5)(11.5^2)$ = 121 in³
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(5.5)(11.5^3)$ = 697 in⁴
$A = b\ d = (5.5)(11.5)$ = 63.25 in²
E = 1600000 psi
C_D = Duration = 1.15
C_F = NDS Table 4D = 1.0
F_{cPerp} = Design compression stress perpendicular to grain = 625 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 625 \times 5.5 \times 0.125 \times 12$ = **5,156 lbs**
F_v = Design shear stress = 170 psi
$V_{All} = F_v\ C_D \times A = (170)(1.15) \times 63.25$ = **12,365 lbs**
F_b = Design bending stress = 1350 psi
$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (1350)(1.15)(1)\left(\frac{121}{12}\right)$ = **15,684 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 206 \times \frac{1}{12}(19.5 \times 12)^4}{(384)(1600000)(697)}$ = 0.602 in

Allowable Live Load Deflection = $\frac{L_{span} \times 12}{360} = \frac{19.5 \times 12}{360}$ = 0.65 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 247 \times \frac{1}{12}(19.5 \times 12)^4}{(384)(1600000)(697)}$ = 0.719 in

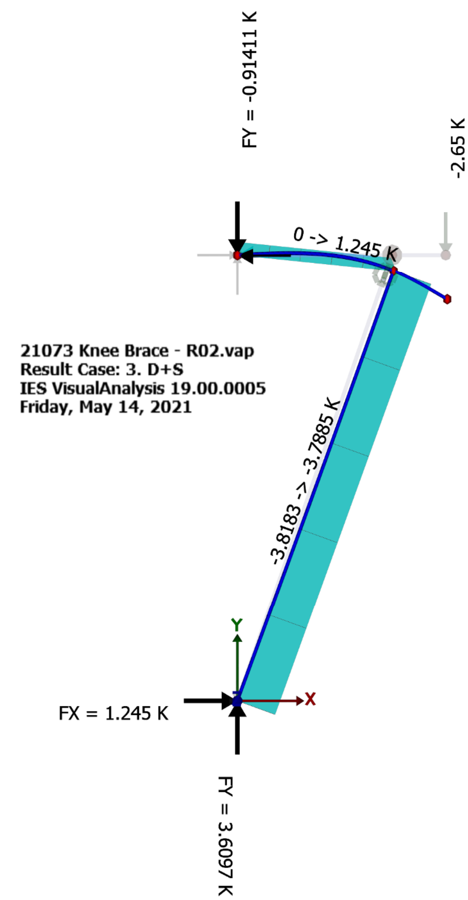
Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{19.5 \times 12}{240}$ = 0.975 in **OK**

Therefore use a 6x12 #1 DF for the beam.

R05 - Knee Brace

$V_D = \frac{W_D}{W} \times V_{Max} = \frac{40.2}{247} \times 2404$ = **391 lbs**

$V_S = \frac{W_L}{W} \times V_{Max} = \frac{206}{247} \times 2404$ = **2,005 lbs**



The 6x6 DF knee brace is acceptable by inspection.

Top Connection

T = Tension = 975 lbs

(2) DTT1Z are acceptable.

6x6 DF Knee Brace Connections

V = Shear = 1,962 lbs

C_D = Duration Factor = 1.15

Z_B = 960 lbs per 3/4” lag screw

$Z_B' = 2 \times Z_B \times C_D = 2 \times 960 \times 1.15$ = 2,208 lbs/ft

Use (2) 3/4” lag screws at the bottom of the knee brace.

Use (1) 5/8” diameter threaded rod w/ epoxy anchor (10” embed) at the top of the knee brace.

R06 - Roof Ledger Connection

L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 10$ = 5 ft

W_D = Dead Load = $L_{Rt} \times R_{DL} \times D_{F1}$ = $5 \times 15 \times 1.02$ = 76.5 plf
W_L = Live Load = $L_{Rt} \times R_{SL}$ = 5×127 = 635 plf
W = Total Load = $W_D + W_L$ = $76.5 + 635$ = 711.5 plf

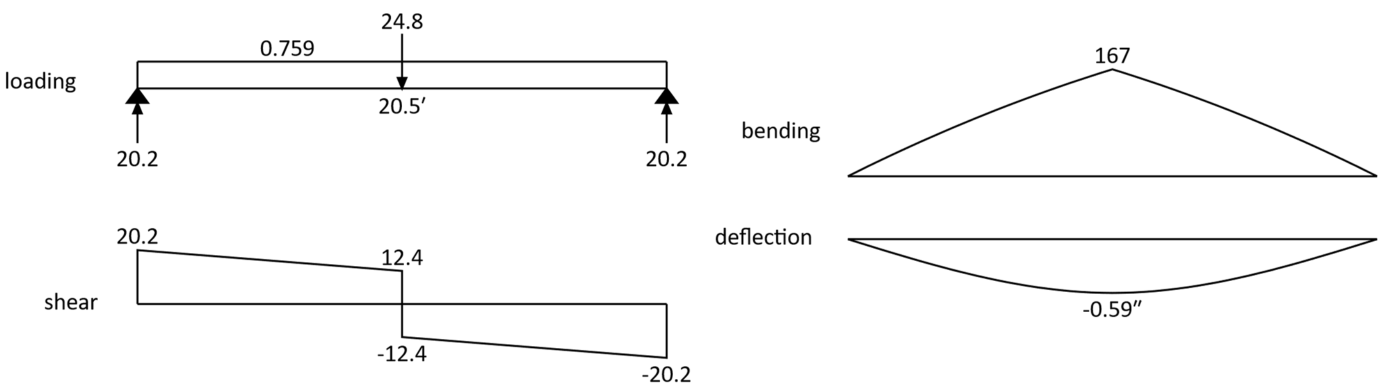
Z_S = 200 lbs per 1/4"x3 1/2" Simpson SDS screw
s_S = Screw Spacing = 1.33 ft
$ZS' = \frac{1}{s_S} 5 \times Z_S\, C_D = \frac{1}{1.33}(5)(200)(1.15)$ = 865 lbs/ft

Therefore: **Use (5) 1/4"x3 1/2" Simpson SDS screws @ 16" o.c.**

S01 - 8 3/4" x 24" GLB Second Floor Beam

b = Width = 8.75 in
d = Depth = 24 in
l_b = Bearing Length (assumed) = 5.5 in = 0.458 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 8.75 \times \frac{1}{12} \times 24$ = 51.0 lb/ft
L_{span} = Length of span = 20.5 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times (15 + 12.5)$ = 13.75 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 51.0 + 13.75 \times 13 + 0 \times 15 \times 1.02 + 0 \times 12$ = 230 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL} = 13.75 \times 40 + 0 \times 127$ = 550 plf
W = Total Load = $W_D + W_L = 230 + 550$ = 780 plf
P = From R01 = 24,800 lbs

Forces



Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6} b\, d^2 = \left(\frac{1}{6}\right)(8.75)(24^2)$ = 840 in³
$I_x = \frac{1}{12} b\, d^3 = \left(\frac{1}{12}\right)(8.75)(24^3)$ = 10,080 in⁴
$A = b\, d = (8.75)(24)$ = 210 in²
E = 1800000 psi
C_D = 1.15
C_V = NDS 5.3.6 = $\min\left(1.0; \left(\frac{21}{L_{span}}\right)^{0.1} \left(\frac{12}{d}\right)^{0.1} \left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0; \left(\frac{21}{20.5}\right)^{0.1} \left(\frac{12}{24}\right)^{0.1} \left(\frac{5.125}{8.75}\right)^{0.1}\right)$ = 0.887

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 650 \times 8.75 \times 0.458 \times 12$ = **31,259 lbs**

F_v = Design shear stress = 265 psi
$V_{All} = F_v\, C_D \times A = (265)(1.15) \times 210$ = **63,998 lbs**

F_b = Design bending stress = 2400 psi
$M_{All} = F_b C_D C_V \left(\frac{S_x}{12}\right) = (2400)(1.15)(0.887)\left(\frac{840}{12}\right)$ = **171,285 ft·lbs**

Deflection

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{20.5 \times 12}{240}$ = 1.025 in **OK**

Therefore use an 8 3/4" x 24" GLB for the beam.

S02 - 6x10 #1 DF Second Floor Beam

b = Width = 5.5 in
d = Depth = 9.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 9.5$ = 12.7 lb/ft
L_{span} = Length of span = 5 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times 1.33$ = 0.665 ft
L_{Rt} = Roof Tributary area = $\frac{1}{2} \times (10 + 10.25) + 1$ = 11.1 ft
h_w = Wall Height = h_U = 6.67 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 12.7+0.665×13 +11.1×15×1.02 +6.67×12 = 272 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = 0.665×40 +11.1×127 = 1,439 plf
W = Total Load = $W_D + W_L$ = 272 + 1,439 = 1,711 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(1{,}711)(5)$ = **4,278 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 1{,}711 \times 5^2$ = **5,347 ft·lbs**

Allowable Forces - #1 DF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(5.5)(9.5^2)$ = 82.7 in³
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(5.5)(9.5^3)$ = 393 in⁴
$A = b\ d = (5.5)(9.5)$ = 52.25 in²
E = 1600000 psi

C_D = Duration = 1.15
C_F = NDS Table 4D = 1.0

F_{cPerp} = Design compression stress perpendicular to grain = 625 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12$ = 625×5.5×0.125×12 = **5,156 lbs**

F_v = Design shear stress = 170 psi
$V_{All} = F_v\ C_D \times A$ = (170)(1.15)×52.25 = **10,215 lbs**

F_b = Design bending stress = 1350 psi
$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (1350)(1.15)(1)\left(\frac{82.7}{12}\right)$ = **10,703 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 1439 \times \frac{1}{12}(5 \times 12)^4}{(384)(1600000)(393)}$ = 0.0322 in

Allowable Live Load Deflection = $\frac{L_{span} \times 12}{360} = \frac{5 \times 12}{360}$ = 0.167 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 1711 \times \frac{1}{12}(5 \times 12)^4}{(384)(1600000)(393)}$ = 0.0383 in

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{5 \times 12}{240}$ = 0.25 in **OK**

Therefore use 6x10 #1 DF for the beam.

S03 - 5 1/2” x 18” GLB Sliding Door Header - Continuous

b = Width = 5.5 in

d = Depth = 18 in

l_b = Bearing Length (assumed) = 3.5 in = 0.292 ft

w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 18$ = 24.1 lb/ft

L_{span} = Length of span = 16.5 ft

L_{ft} = Floor Tributary area = $\frac{1}{2} \times 15$ = 7.5 ft

L_{Rt} = Roof Tributary area = $\frac{1}{2} \times 2 + 1$ = 2 ft

h_w = Wall Height = h_U = 6.67 ft

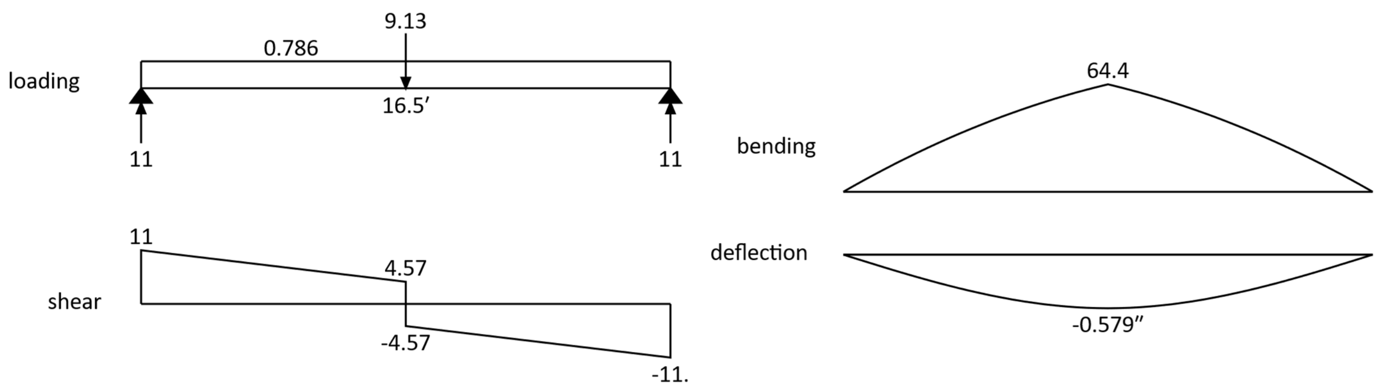
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = $24.1 + 7.5 \times 13 + 2 \times 15 \times 1.02 + 6.67 \times 12$ = 232 plf

W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = $7.5 \times 40 + 2 \times 127$ = 554 plf

W = Total Load = $W_D + W_L$ = $232 + 554$ = 786 plf

P = From R01 = 9,130 lbs

Forces



Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6} b\, d^2 = \left(\frac{1}{6}\right)(5.5)(18^2)$ = 297 in³

$I_x = \frac{1}{12} b\, d^3 = \left(\frac{1}{12}\right)(5.5)(18^3)$ = 2,673 in⁴

$A = b\, d = (5.5)(18)$ = 99 in²

E = 1800000 psi

C_D = 1.15

C_V = NDS 5.3.6 = $\min\left(1.0; \left(\frac{21}{L_{span}}\right)^{0.1} \left(\frac{12}{d}\right)^{0.1} \left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0; \left(\frac{21}{16.5}\right)^{0.1} \left(\frac{12}{18}\right)^{0.1} \left(\frac{5.125}{5.5}\right)^{0.1}\right)$ = 0.977

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi

R_{All} = $\boldsymbol{F_{cPerp} \times b \times l_b \times 12}$ = $650 \times 5.5 \times 0.292 \times 12$ = **12,527 lbs**

F_v = Design shear stress = 265 psi

V_{All} = $\boldsymbol{F_v C_D \times A}$ = $(265)(1.15) \times 99$ = **30,170 lbs**

F_b = Design bending stress = 2400 psi

M_{All} = $\boldsymbol{F_b C_D C_V \left(\frac{S_x}{12}\right)} = \boldsymbol{(2400)(1.15)(0.977)\left(\frac{297}{12}\right)}$ = **66,724 ft·lbs**

Deflection

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{16.5 \times 12}{240}$ = 0.825 in **OK**

Therefore use a 5 1/2” x 18” GLB for the header with 4x6 #2 DF supports.

M01 - 3 1/2” x 11 7/8” PSL Floor Beam

l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = 13 lb/ft
L_{span} = Length of span = 5.5 ft
L_{ft} = Floor Tributary area $= \frac{1}{2} \times (4.5 + 4.5) = 4.5$ ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 13 + 4.5 \times 13 + 0 \times 15 \times 1.02 + 0 \times 12 = 71.5$ plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL} = 4.5 \times 40 + 0 \times 127 = 180$ plf
W = Total Load = $W_D + W_L = 71.5 + 180 = 251.5$ plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(251.5)(5.5) =$ **692 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 251.5 \times 5.5^2 =$ **951 ft·lbs**

Allowable Forces - 2.0E Parallam from Trus Joist Tables

V_{Allow} = 8,035 lbs **OK**

M_{Allow} = 19,900 ft-lbs **OK**

Deflection

$I_x = 488$ in^4
E = 2000000 psi

$$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 180 \times \frac{1}{12}(5.5 \times 12)^4}{(384)(2000000)(488)} = 0.00380 \text{ in}$$

$$\text{Allowable Live Load Deflection} = \frac{L_{span} \times 12}{360} = \frac{5.5 \times 12}{360} = 0.183 \text{ in}$$ **OK**

$$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 251.5 \times \frac{1}{12}(5.5 \times 12)^4}{(384)(2000000)(488)} = 0.00531 \text{ in}$$

$$\text{Allowable Total Deflection} = \frac{L_{span} \times 12}{240} = \frac{5.5 \times 12}{240} = 0.275 \text{ in}$$ **OK**

Therefore a 3 1/2” x 11 7/8” parallam is OK for this beam.

M02 - 3 1/2" x 11 7/8" PSL Floor Beam

l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = 13 lb/ft
L_{span} = Length of span = 10.5 ft
L_{ft} = Floor Tributary area $= \frac{1}{2} \times (10.5 + 2 + 10.5 + 10.5) = 16.75$ ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = $h_U + h_M$ = 6.67+9.1 = 15.8 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 13+16.75×13 +0×15×1.02 +15.8×12 = 420 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = 16.75×40 +0×127 = 670 plf
W = Total Load = $W_D + W_L$ = 420 + 670 = 1,090 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(1{,}090)(10.5)$ = **5,722 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 1{,}090 \times 10.5^2$ = **15,021 ft·lbs**

Allowable Forces - 2.0E Parallam from Trus Joist Tables

V_{Allow} = 8,035 lbs **OK**

M_{Allow} = 19,900 ft-lbs **OK**

Deflection

$I_x = 488\ in^4$
E = 2000000 psi

$$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 670 \times \frac{1}{12}(10.5 \times 12)^4}{(384)(2000000)(488)} = 0.188 \text{ in}$$

$$\text{Allowable Live Load Deflection} = \frac{L_{span} \times 12}{360} = \frac{10.5 \times 12}{360} = 0.35 \text{ in}$$ **OK**

$$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 1090 \times \frac{1}{12}(10.5 \times 12)^4}{(384)(2000000)(488)} = 0.305 \text{ in}$$

$$\text{Allowable Total Deflection} = \frac{L_{span} \times 12}{240} = \frac{10.5 \times 12}{240} = 0.525 \text{ in}$$ **OK**

Therefore a 3 1/2" x 11 7/8" parallam is OK for this beam with HGUS412 hanger.

M03 - 3 1/2” x 11 7/8” PSL Floor Beam

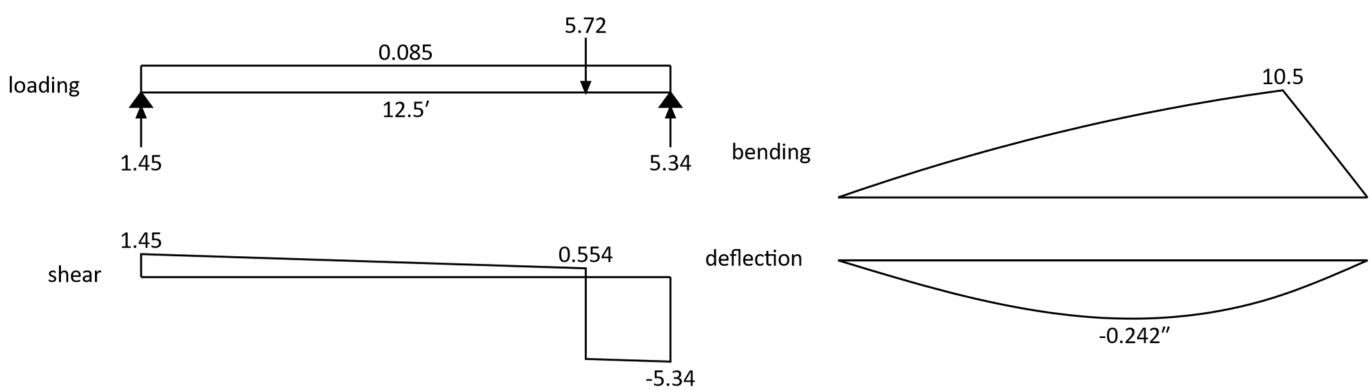
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = 13 lb/ft
L_{span} = Length of span = 12.5 ft

L_{ft} = Floor Tributary area = 1.33 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = $13 + 1.33 \times 13 + 0 \times 15 \times 1.02 + 0 \times 12$ = 30.3 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = $1.33 \times 40 + 0 \times 127$ = 53.2 plf
W = Total Load = $W_D + W_L$ = $30.3 + 53.2$ = 83.5 plf

P = From M02 = 5,722 lbs

Forces



Allowable Forces - 2.0E Parallam from Trus Joist Tables

V_{Allow} = 8,035 lbs **OK**

M_{Allow} = 19,900 ft-lbs **OK**

Deflection

I_x = 488 in^4
E = 2000000 psi

$Allowable\ Total\ Deflection = \frac{L_{span} \times 12}{240} = \frac{10.5 \times 12}{240} = 0.525$ in **OK**

Therefore a 3 1/2” x 11 7/8” parallam is OK for this beam with HU412 hanger.

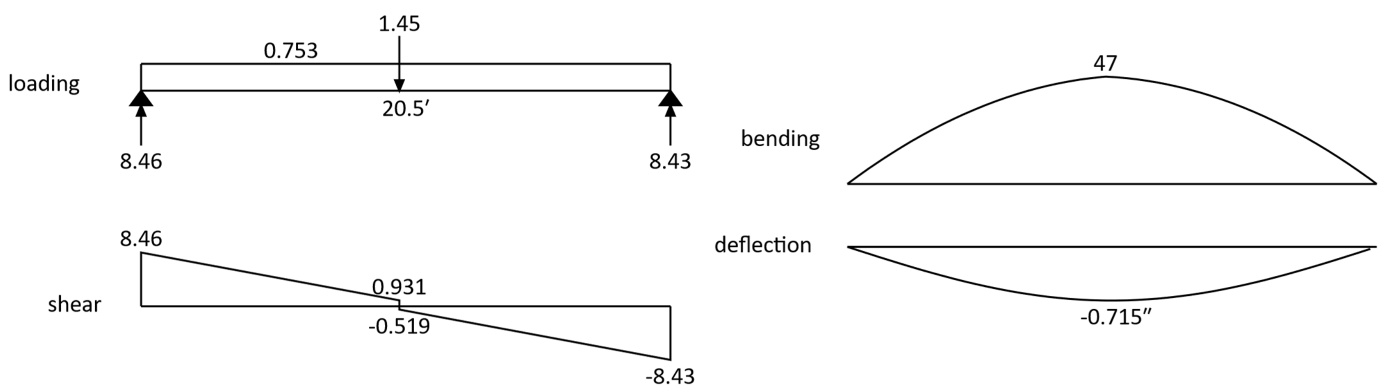
M04 - 5 1/2" x 18" GLB Second Floor Beam

b = Width = 5.5 in
d = Depth = 18 in
l_b = Bearing Length (assumed) = 2.75 in = 0.229 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 18$ = 24.1 lb/ft
L_{span} = Length of span = 20.5 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times (15 + 12.5) = 13.75$ ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 24.1 + 13.75 \times 13 + 0 \times 15 \times 1.02 + 0 \times 12$ = 203 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL} = 13.75 \times 40 + 0 \times 127$ = 550 plf
W = Total Load = $W_D + W_L = 203 + 550$ = 753 plf

P = From M03 = 1,450 lbs

Forces



Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6} b\, d^2 = \left(\frac{1}{6}\right)(5.5)(18^2)$ = 297 in³
$I_x = \frac{1}{12} b\, d^3 = \left(\frac{1}{12}\right)(5.5)(18^3)$ = 2,673 in⁴
$A = b\, d = (5.5)(18)$ = 99 in²
E = 1800000 psi

C_D = 1.15

C_V = NDS 5.3.6 = $\min\left(1.0; \left(\frac{21}{L_{span}}\right)^{0.1} \left(\frac{12}{d}\right)^{0.1} \left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0; \left(\frac{21}{20.5}\right)^{0.1} \left(\frac{12}{18}\right)^{0.1} \left(\frac{5.125}{5.5}\right)^{0.1}\right)$ = 0.956

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi
R_{All} **=** $\boldsymbol{F_{cPerp} \times b \times l_b \times 12}$ **=** $650 \times 5.5 \times 0.229 \times 12$ = **9,824 lbs**

F_v = Design shear stress = 265 psi
V_{All} **=** $\boldsymbol{F_v\, C_D \times A}$ **=** $(265)(1.15) \times 99$ = **30,170 lbs**

F_b = Design bending stress = 2400 psi
M_{All} **=** $\boldsymbol{F_b C_D C_V \left(\frac{S_x}{12}\right)}$ **=** $\boldsymbol{(2400)(1.15)(0.956)\left(\frac{297}{12}\right)}$ = **65,291 ft·lbs**

Deflection

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{20.5 \times 12}{240}$ = 1.025 in **OK**

Therefore use a 5 1/2" x 18" GLB for the beam with HGUS5.50/14 hanger.

M05 - 3 1/2" x 11 7/8" PSL Floor Beam

l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = 13 lb/ft
L_{span} = Length of span = 10 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times (13.67 + 12.5)$ = 13.1 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = $13+13.1\times13+0\times15\times1.02+0\times12$ = 183 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = $13.1\times40+0\times127$ = 523 plf
W = Total Load = $W_D + W_L$ = $183 + 523$ = 707 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(707)(10)$ = **3,533 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 707 \times 10^2$ = **8,831 ft·lbs**

Allowable Forces - 2.0E Parallam from Trus Joist Tables

V_{Allow} = 8,035 lbs **OK**

M_{Allow} = 19,900 ft-lbs **OK**

Deflection

I_x = 488 in^4
E = 2000000 psi

$$\Delta_L = \frac{5\times W_L\times\frac{1}{12}\times(L_{span}\times12)^4}{384\,E\,I_x} = \frac{5\times523\times\frac{1}{12}(10\times12)^4}{(384)(2000000)(488)} = 0.121 \text{ in}$$

$$Allowable\ Live\ Load\ Deflection = \frac{L_{span}\times12}{360} = \frac{10\times12}{360} = 0.333 \text{ in}$$ **OK**

$$\Delta_{Max} = \frac{5\times W\times\frac{1}{12}\times(L_{span}\times12)^4}{384\,E\,I_x} = \frac{5\times707\times\frac{1}{12}(10\times12)^4}{(384)(2000000)(488)} = 0.163 \text{ in}$$

$$Allowable\ Total\ Deflection = \frac{L_{span}\times12}{240} = \frac{10\times12}{240} = 0.5 \text{ in}$$ **OK**

Therefore a 3 1/2" x 11 7/8" parallam is OK for this beam with HGUS412 hanger.

M06 - 8 3/4” x 18” GLB Main Floor Beam

b = Width = 8.75 in
d = Depth = 18 in
l_b = Bearing Length (assumed) = 7.5 in = 0.625 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d$ = $35 \times \frac{1}{12} \times 8.75 \times \frac{1}{12} \times 18$ = 38.3 lb/ft
L_{span} = Length of span = 12.5 ft

L_{ft} = Floor Tributary area = $1.33 + \frac{1}{2} \times 1.33$ = 1.995 ft
L_{Rt} = Roof Tributary area = $\frac{1}{2}(10 + 10.5) + 1$ = 11.25 ft
h_w = Wall Height = h_U+h_M = 6.67+9.1 = 15.8 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 38.3+1.995×13 +11.25×15×1.02+15.8×12 = 426 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = 1.995×40 +11.25×127 = 1,509 plf
W = Total Load = $W_D + W_L$ = 426 + 1,509 = 1,934 plf

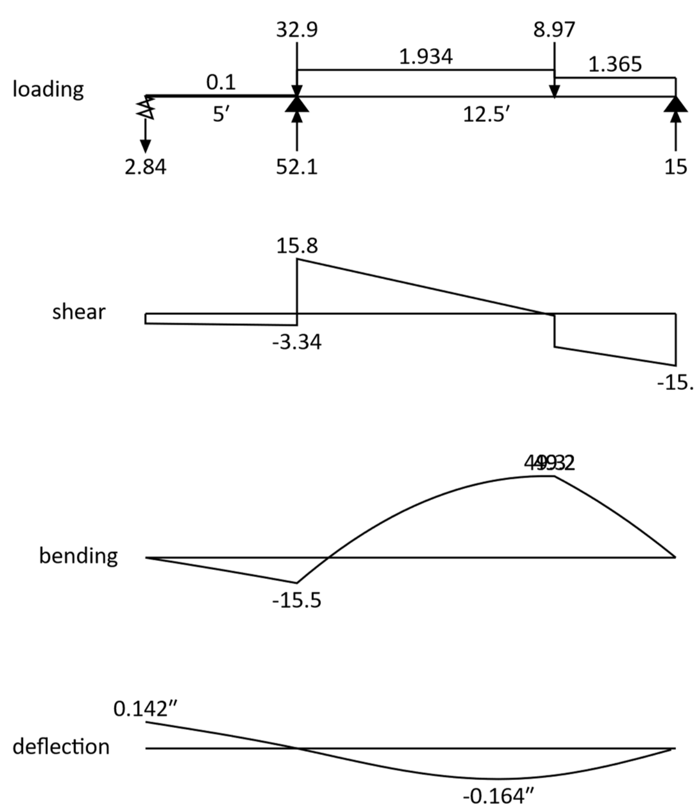
L_{ft} = Floor Tributary area = $1.33 + \frac{1}{2} \times 1.33$ = 1.995 ft
L_{Rt} = Roof Tributary area = $\frac{1}{2}(2 + 10.5) + 1$ = 7.25 ft
h_w = Wall Height = h_U+h_M = 6.67+9.1 = 15.8 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 38.3+1.995×13 +7.25×15×1.02+15.8×12 = 364 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{Rt} \times R_{SL}$ = 1.995×40 +7.25×127 = 1,001 plf
W = Total Load = $W_D + W_L$ = 364 + 1,001 = 1,365 plf

P = From S01+S02+M04 = 20,200+4,278+8,460 = 32,938 lbs

P = From GT+M05 = $3533 + \frac{1}{2} \times 10 \times \left(\frac{1}{2} \times 13 + 1\right)(R_{DL} \times D_{F2} + R_{SL})$
= $3533 + \frac{1}{2} \times 10 \times \left(\frac{1}{2} \times 13 + 1\right)(15 \times 1.2 + 127)$ = 8,971 lbs

Forces



Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6}\, b\, d^2 = \left(\frac{1}{6}\right)(8.75)(18^2) = 472.5\ \text{in}^3$

$I_x = \frac{1}{12}\, b\, d^3 = \left(\frac{1}{12}\right)(8.75)(18^3) = 4{,}252\ \text{in}^4$

$A = b\, d = (8.75)(18) = 157.5\ \text{in}^2$

E = 1800000 psi

$C_D = 1.15$

$C_V = \text{NDS 5.3.6} = \min\left(1.0;\left(\frac{21}{L_{span}}\right)^{0.1}\left(\frac{12}{d}\right)^{0.1}\left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0;\left(\frac{21}{12.5}\right)^{0.1}\left(\frac{12}{18}\right)^{0.1}\left(\frac{5.125}{8.75}\right)^{0.1}\right) = 0.959$

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi

$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 650 \times 8.75 \times 0.625 \times 12 =$ **42,656 lbs** **Use CC98 Cap**

F_v = Design shear stress = 265 psi

$V_{All} = F_v\, C_D \times A = (265)(1.15) \times 157.5 =$ **47,998 lbs**

F_b = Design bending stress = 1850 psi

$M_{All} = F_b C_D C_V \left(\frac{S_x}{12}\right) = (1850)(1.15)(0.959)\left(\frac{472.5}{12}\right) =$ **80,336 ft·lbs**

Deflection

$\text{Allowable Total Deflection} = \frac{L_{span} \times 12}{240} = \frac{12.5 \times 12}{240} = 0.625\ \text{in}$ **OK**

Therefore use an 8 3/4” x 18” GLB for the beam with 8x10 #1 DF post in the center.

M07 - 5 1/2" x 15" GLB Garage Door Header - Continuous

b = Width = 5.5 in
d = Depth = 15 in
l_b = Bearing Length (assumed) = 3.5 in = 0.292 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 15$ = 20.1 lb/ft
L_{span} = Length of span = 16.5 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times 15$ = 7.5 ft
L_{ftD} = Deck Tributary area = $\frac{1}{2} \times 8$ = 4 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = h_M = 9.1 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DL} + L_{ftD} \times F_{DLD} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 20.1+ 7.5×13+4×9+0×15×1.02 +9.1×12 = 263 plf
W_L = Live Load = $L_{ft} \times F_{LL} + L_{ftD} \times \max(F_{LDH}; R_{SL}) + L_{Rt} \times R_{SL}$ = 7.5×40+4×max(120;127)+0×127 = 808 plf
W = Total Load = $W_D + W_L$ = 263 + 808 = 1,071 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(1{,}071)(16.5)$ = **8,834 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 1{,}071 \times 16.5^2$ = **36,441 ft·lbs**

Allowable Forces - 24F-V4 DF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(5.5)(15^2)$ = 206 in³
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(5.5)(15^3)$ = 1,547 in⁴
$A = b\ d = (5.5)(15)$ = 82.5 in²
E = 1800000 psi
C_D = 1.15

$$C_V = \text{NDS 5.3.6} = \min\left(1.0; \left(\frac{21}{L_{span}}\right)^{0.1} \left(\frac{12}{d}\right)^{0.1} \left(\frac{5.125}{b}\right)^{0.1}\right) = \min\left(1.0; \left(\frac{21}{16.5}\right)^{0.1} \left(\frac{12}{15}\right)^{0.1} \left(\frac{5.125}{5.5}\right)^{0.1}\right) = 0.995$$

F_{cPerp} = Design compression stress perpendicular to grain = 650 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 650 \times 5.5 \times 0.292 \times 12$ = **12,513 lbs**
F_v = Design shear stress = 265 psi
$V_{All} = F_v\ C_D \times A = (265)(1.15) \times 82.5$ = **25,142 lbs**
F_b = Design bending stress = 2400 psi
$M_{All} = F_b C_D C_V \left(\frac{S_x}{12}\right) = (2400)(1.15)(0.995)\left(\frac{206}{12}\right)$ = **47,189 ft·lbs**

Deflection

$$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 808 \times \frac{1}{12}(16.5 \times 12)^4}{(384)(1800000)(1547)} = 0.484 \text{ in}$$

$$\text{Allowable Live Load Deflection} = \frac{L_{span} \times 12}{360} = \frac{16.5 \times 12}{360} = 0.55 \text{ in}$$ **OK**

$$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 1071 \times \frac{1}{12}(16.5 \times 12)^4}{(384)(1800000)(1547)} = 0.641 \text{ in}$$

$$\text{Allowable Total Deflection} = \frac{L_{span} \times 12}{240} = \frac{16.5 \times 12}{240} = 0.825 \text{ in}$$ **OK**

Therefore use a 5 1/2" x 15" GLB for the header.

M08 - 2x10 P.T. #2 HF Deck Joists

b = Width = 1.5 in
d = Depth = 9.25 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 1.5 \times \frac{1}{12} \times 9.25$ = 3.37 lb/ft
L_{span} = Length of span = 8 ft
L_{ft} = Floor Tributary area = 1.0 ft
W_D = Dead Load = $L_{ft} \times F_{DLD} = 1 \times 9$ = 9 plf
W_L = Live Load = $L_{ft} \times \max(F_{LDH};R_{SL}) = 1 \times \max(120;127)$ = 127 plf
W = Total Load = $W_D + W_L = 9 + 127$ = 136 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(136)(8)$ = **544 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 136 \times 8^2$ = **1,088 ft·lbs**

Allowable Forces - #2 HF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(1.5)(9.25^2)$ = 21.4 in³
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(1.5)(9.25^3)$ = 98.9 in⁴
$A = b\ d = (1.5)(9.25)$ = 13.9 in²
E = 1300000 psi
C_D = Duration = 1.0
C_F = NDS Table 4A = 1.1
C_r = Repetitive = 1.15

F_{cPerp} = Design compression stress perpendicular to grain = 405 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 405 \times 1.5 \times 0.125 \times 12$ = **911 lbs**

F_v = Design shear stress = 150 psi
$V_{All} = F_v\ C_D \times A = (150)(1) \times 13.9$ = **2,081 lbs**

F_b = Design bending stress = 850 psi
$M_{All} = F_b C_D C_F\ C_r \left(\frac{S_x}{12}\right) = (850)(1)(1.1)(1.15)\left(\frac{21.4}{12}\right)$ = **1,917 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 127 \times \frac{1}{12}(8 \times 12)^4}{(384)(1300000)(98.9)}$ = 0.0910 in

$Allowable\ Live\ Load\ Deflection = \frac{L_{span} \times 12}{360} = \frac{8 \times 12}{360}$ = 0.267 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 136 \times \frac{1}{12}(8 \times 12)^4}{(384)(1300000)(98.9)}$ = 0.0975 in

$Allowable\ Total\ Deflection = \frac{L_{span} \times 12}{240} = \frac{8 \times 12}{240}$ = 0.4 in **OK**

Therefore use 2x10 P.T. #2 HF deck joists @ 12” o.c. with LUS210 or JB210A hangers.

Deck Ledger Connection

V = Shear = $V_{Max}\frac{12}{16} = (544)\left(\frac{12}{16}\right)$ = 408 lbs/ft
Z_S = 200 lbs per 1/4"x3 1/2" Simpson SDS screw
s_S = Screw Spacing = 1.0 ft
$ZS' = \frac{1}{s_S} 3 \times Z_S\ C_D = \frac{1}{1}(3)(200)(1)$ = 600 lbs/ft

Therefore: **Use (3) 1/4”x3 1/2" Simpson SDS screws @ 12” o.c.**

M10 - 6x12 P.T. #2 HF Deck Beam

b = Width = 5.5 in
d = Depth = 11.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 5.5 \times \frac{1}{12} \times 11.5$ = 15.4 lb/ft
L_{span} = Length of span = 10 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times (6+4)$ = 5 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DLD} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 15.4+5×9 +0×15×1.02 +0×12 = 60.4 plf
W_L = Live Load = $L_{ft} \times F_{LLD} + L_{Rt} \times R_{SL}$ = 5×60 +0×127 = 300 plf
W = Total Load = $W_D + W_L$ = 60.4 + 300 = 360 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(360)(10)$ = **1,802 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 360 \times 10^2$ = **4,505 ft·lbs**

Allowable Forces - #2 HF

$S_x = \frac{1}{6} b\ d^2 = \left(\frac{1}{6}\right)(5.5)(11.5^2)$ = 121 in^3
$I_x = \frac{1}{12} b\ d^3 = \left(\frac{1}{12}\right)(5.5)(11.5^3)$ = 697 in^4
$A = b\ d = (5.5)(11.5)$ = 63.25 in^2
E = 1100000 psi

C_D = Duration = 1.0
C_F = NDS Table 4D = 1.0
F_{cPerp} = Design compression stress perpendicular to grain = 405 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12$ = 405×5.5×0.125×12 = **3,341 lbs**

F_v = Design shear stress = 140 psi
$V_{All} = F_v\ C_D \times A$ = (140)(1)×63.25 = **8,855 lbs**

F_b = Design bending stress = 675 psi
$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (675)(1)(1)\left(\frac{121}{12}\right)$ = **6,819 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 300 \times \frac{1}{12}(10 \times 12)^4}{(384)(1100000)(697)}$ = 0.0880 in

Allowable Live Load Deflection = $\frac{L_{span} \times 12}{360} = \frac{10 \times 12}{360}$ = 0.333 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 360 \times \frac{1}{12}(10 \times 12)^4}{(384)(1100000)(697)}$ = 0.106 in

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{10 \times 12}{240}$ = 0.5 in **OK**

Therefore use a 6x12 P.T. #2 HF for the beam.

M11 - 4x12 P.T. #2 HF Stair Stringers

b = Width = 11.25 in
d = Depth = 3.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 11.25 \times \frac{1}{12} \times 3.5$ = 9.57 lb/ft
L_{span} = Length of span = 5.5 ft
L_{ft} = Floor Tributary area = 1 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft

W_D = Dead Load = $w_s + L_{ft} \times F_{DLD} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL}$ = 9.57+1×9 +0×15×1.02 +0×12 = 18.6 plf
W_L = Live Load = $L_{ft} \times F_{LLD} + L_{Rt} \times R_{SL}$ = 1×60 +0×127 = 60 plf
W = Total Load = $W_D + W_L$ = 18.6 + 60 = 78.6 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(78.6)(5.5)$ = **216 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 78.6 \times 5.5^2$ = **297 ft·lbs**

V_{MaxP} = **300 lbs**
$M_{MaxP} = \frac{1}{4} \times 300 \times L_{span} = \frac{1}{4} \times 300 \times 5.5$ = **412.5 ft·lbs**

Allowable Stresses - #2 HF

$S_x = \frac{1}{6}\,b\,d^2 = \left(\frac{1}{6}\right)(11.25)(3.5^2)$ = 23.0 in³
$I_x = \frac{1}{12}\,b\,d^3 = \left(\frac{1}{12}\right)(11.25)(3.5^3)$ = 40.2 in⁴
$A = b\,d = (11.25)(3.5)$ = 39.4 in²
E = 1300000 psi
C_D = Duration = 1.0
C_F = NDS Table 4A = 1.1
C_{fu} = NDS Table 4A = 1.1

F_{cPerp} = Design compression stress perpendicular to grain = 405 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12$ = 405×11.25×0.125×12 = **6,834 lbs**

F_v = Design shear stress = 150 psi
$V_{All} = F_v\,C_D \times A$ = (150)(1)×39.4 = **5,906 lbs**

F_b = Design bending stress = 850 psi
$M_{All} = F_b C_D C_F C_{fu}\left(\frac{S_x}{12}\right) = (850)(1)(1.1)(1.1)\left(\frac{23.0}{12}\right)$ = **1,969 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\,E\,I_x} = \frac{5 \times 60 \times \frac{1}{12}(5.5 \times 12)^4}{(384)(1300000)(40.2)}$ = 0.0236 in

$Allowable\ Live\ Load\ Deflection = \frac{L_{span} \times 12}{360} = \frac{5.5 \times 12}{360}$ = 0.183 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\,E\,I_x} = \frac{5 \times 78.6 \times \frac{1}{12}(5.5 \times 12)^4}{(384)(1300000)(40.2)}$ = 0.0310 in

$Allowable\ Total\ Deflection = \frac{L_{span} \times 12}{240} = \frac{5.5 \times 12}{240}$ = 0.275 in **OK**

Therefore use a 4x12 P.T. #2 HF for the treads with TA10Z hangers.

M12 - 4x14 P.T. #2 HF Stair Stringers

b = Width = 3.5 in
d = Depth = 13.5 in
l_b = Bearing Length (assumed) = 1.5 in = 0.125 ft
w_s = Self Weight = $35 \times \frac{1}{12} \times b \times \frac{1}{12} \times d = 35 \times \frac{1}{12} \times 3.5 \times \frac{1}{12} \times 13.5$ = 11.5 lb/ft
L_{span} = Length of span = 14 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times 5.5$ = 2.75 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_s + L_{ft} \times F_{DLD} + L_{Rt} \times R_{DL} \times D_{F1} + h_w \times W_{DL} = 11.5 + 2.75 \times 9 + 0 \times 15 \times 1.02 + 0 \times 12$ = 36.2 plf
W_L = Live Load = $L_{ft} \times F_{LLD} + L_{Rt} \times R_{SL} = 2.75 \times 60 + 0 \times 127$ = 165 plf
W = Total Load = $W_D + W_L = 36.2 + 165$ = 201 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(201)(14)$ = **1,407 lbs**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 201 \times 14^2$ = **4,925 ft·lbs**

Allowable Stresses - #2 HF

$S_x = \frac{1}{6}\, b\, d^2 = \left(\frac{1}{6}\right)(3.5)(13.5^2)$ = 106 in³
$I_x = \frac{1}{12}\, b\, d^3 = \left(\frac{1}{12}\right)(3.5)(13.5^3)$ = 718 in⁴
$A = b\, d = (3.5)(13.5)$ = 47.25 in²
E = 1300000 psi
C_D = Duration = 1.0
C_F = NDS Table 4A = 1.0
F_{cPerp} = Design compression stress perpendicular to grain = 405 psi
$R_{All} = F_{cPerp} \times b \times l_b \times 12 = 405 \times 3.5 \times 0.125 \times 12$ = **2,126 lbs**
F_v = Design shear stress = 150 psi
$V_{All} = F_v\, C_D \times A = (150)(1) \times 47.25$ = **7,088 lbs**
F_b = Design bending stress = 850 psi
$M_{All} = F_b C_D C_F \left(\frac{S_x}{12}\right) = (850)(1)(1)\left(\frac{106}{12}\right)$ = **7,530 ft·lbs**

Deflection

$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 165 \times \frac{1}{12}(14 \times 12)^4}{(384)(1300000)(718)}$ = 0.153 in

Allowable Live Load Deflection = $\frac{L_{span} \times 12}{360} = \frac{14 \times 12}{360}$ = 0.467 in **OK**

$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384\ E\ I_x} = \frac{5 \times 201 \times \frac{1}{12}(14 \times 12)^4}{(384)(1300000)(718)}$ = 0.186 in

Allowable Total Deflection = $\frac{L_{span} \times 12}{240} = \frac{14 \times 12}{240}$ = 0.7 in **OK**

Therefore use a 4x14 P.T. #2 HF for the stringers with LSSR410Z hangers.

M13 - W10x33 Steel Support Beams

Section = "**W10x33**"
L_{span} = Length of span = 20.5 ft
L_{ft} = Floor Tributary area = $\frac{1}{2} \times 8$ = 4 ft
L_{Rt} = Roof Tributary area = 0 ft
h_w = Wall Height = 0 ft
W_D = Dead Load = $w_{self}+L_{ft}\times F_{DLD} + L_{Rt}\times R_{DL}\times D_{F1} + h_w\times W_{DL}$ = $33+4\times9 +0\times15\times1.02 +0\times12$ = 69 plf
W_L = Live Load = $L_{ft}\times \max(F_{LDH};R_{SL}) + L_{Rt}\times R_{SL}$ = $4\times\max(120;127) +0\times127$ = 508 plf
W = Total Load = $W_D + W_L$ = $69 + 508$ = 577 plf

Forces

$V_{Max} = \frac{1}{2} \times W\ L_{span} = \frac{1}{2}(577)(20.5)$ = 5,914 lbs = **5.91 kips**
$M_{Max} = \frac{1}{8} \times W \times L_{span}^2 = \frac{1}{8} \times 577 \times 20.5^2$ = 30,311 ft·lbs = **30.3 ft·kips**

Allowable Forces

$h = d$ = 9.73 in;
b_f = 7.96 in;
S_x = 35 in^3;
Z_x = 38.8 in^3;
I_x = 171 in^4;
r_x = 4.19 in;
$A_w = h\times t_w = 9.73\times0.29$ = 2.82 in^2;
J = 0.583 in^4;
C_b = 1.0;
Ω_b = 1.67;
L_b = 2.67 ft
t_w = 0.29 in
t_f = 0.435 in
S_y = 9.2 in^3
Z_y = 14 in^3
I_y = 36.6 in^4
r_y = 1.94 in
C_w = 791 in^6
h_o = 9.30 in
E = 29,000 ksi
Ω_v = 1.67

$L_p = 1.76 \times r_y \times \sqrt{\frac{E_S}{F_y}} = 1.76 \times 1.94 \times \sqrt{\frac{29000}{50}}$ = 82.2 in = 6.85 ft

$r_{ts} = \sqrt{\frac{\sqrt{I_y\times C_w}}{S_x}} = \sqrt{\frac{\sqrt{36.6\times791}}{35}}$ = 2.20

c = 1

$$L_r = \frac{1}{12} \times 1.95 \times r_{ts} \times \frac{E}{0.7\times F_y} \times \sqrt{\frac{J\times c}{S_x\times h_o}} \times \sqrt{1 + \sqrt{1 + 6.76 \times \left(\frac{0.7\times F_y}{E} \times \frac{S_x\times h_o}{J\times c}\right)^2}}$$

$$= \frac{1}{12} \times 1.95 \times 2.20 \times \frac{29000}{0.7\times50} \times \sqrt{\frac{0.583\times1}{35\times9.3}} \times \sqrt{1 + \sqrt{1 + 6.76 \times \left(\frac{0.7\times50}{29000}\frac{35\times9.3}{0.583\times1}\right)^2}} = 21.8 \text{ ft}$$

$M_p = F_y \times Z_x = 50\times38.8$ = 1,940 ft·k

$M_a = \frac{M_p}{12\times\Omega_b} = \frac{1940}{12\times1.67}$ = 96.8 **ft·k** **OK**

$\frac{h}{t_w} = \frac{9.73}{0.29}$ = 33.6

$2.24 \times \sqrt{\frac{E}{F_y}} = 2.24 \times \sqrt{\frac{29000}{50}}$ = 53.9

C_v = 1.0

$V_a = \frac{0.6\ F_y A_w C_v}{\Omega_v} = \frac{(0.6)(50)(2.82)(1)}{1.67}$ = 50.7 **kips** **OK**

Deflection

$$\Delta_L = \frac{5 \times W_L \times \frac{1}{12} \times (L_{span} \times 12)^4}{384000\ E\ I_x} = \frac{5 \times 508 \times \frac{1}{12}\ (20.5 \times 12)^4}{(384000)(29000)(171)} = 0.407 \text{ in}$$

$$Allowable\ Live\ Load\ Deflection = \frac{L_{span} \times 12}{360} = \frac{20.5 \times 12}{360} = 0.683 \text{ in}$$ **OK**

$$\Delta_{Max} = \frac{5 \times W \times \frac{1}{12} \times (L_{span} \times 12)^4}{384000\ E\ I_x} = \frac{5 \times 577 \times \frac{1}{12}\ (20.5 \times 12)^4}{(384000)(29000)(171)} = 0.462 \text{ in}$$

$$Allowable\ Total\ Deflection = \frac{L_{span} \times 12}{240} = \frac{20.5 \times 12}{240} = 1.025 \text{ in}$$ **OK**

Therefore a W10x33 steel beam is acceptable.

Floor Joists

L/480 Live Load Deflection

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

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

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	N.A.	N.A.	N.A.	15'-4"	N.A.	N.A.	16'-0"	12'-9"
210	N.A.	N.A.	21'-4"	17'-0"	N.A.	21'-4"	17'-9"	14'-2"
230	N.A.	N.A.	N.A.	19'-2"	N.A.	N.A.	19'-11"	15'-11"
360	N.A.	N.A.	24'-5"	19'-6"	N.A.	24'-5"	20'-4"	16'-3"
560	N.A.	N.A.	29'-10"	23'-10"	N.A.	29'-10"	24'-10"	19'-10"

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

How to Use These Tables

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

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

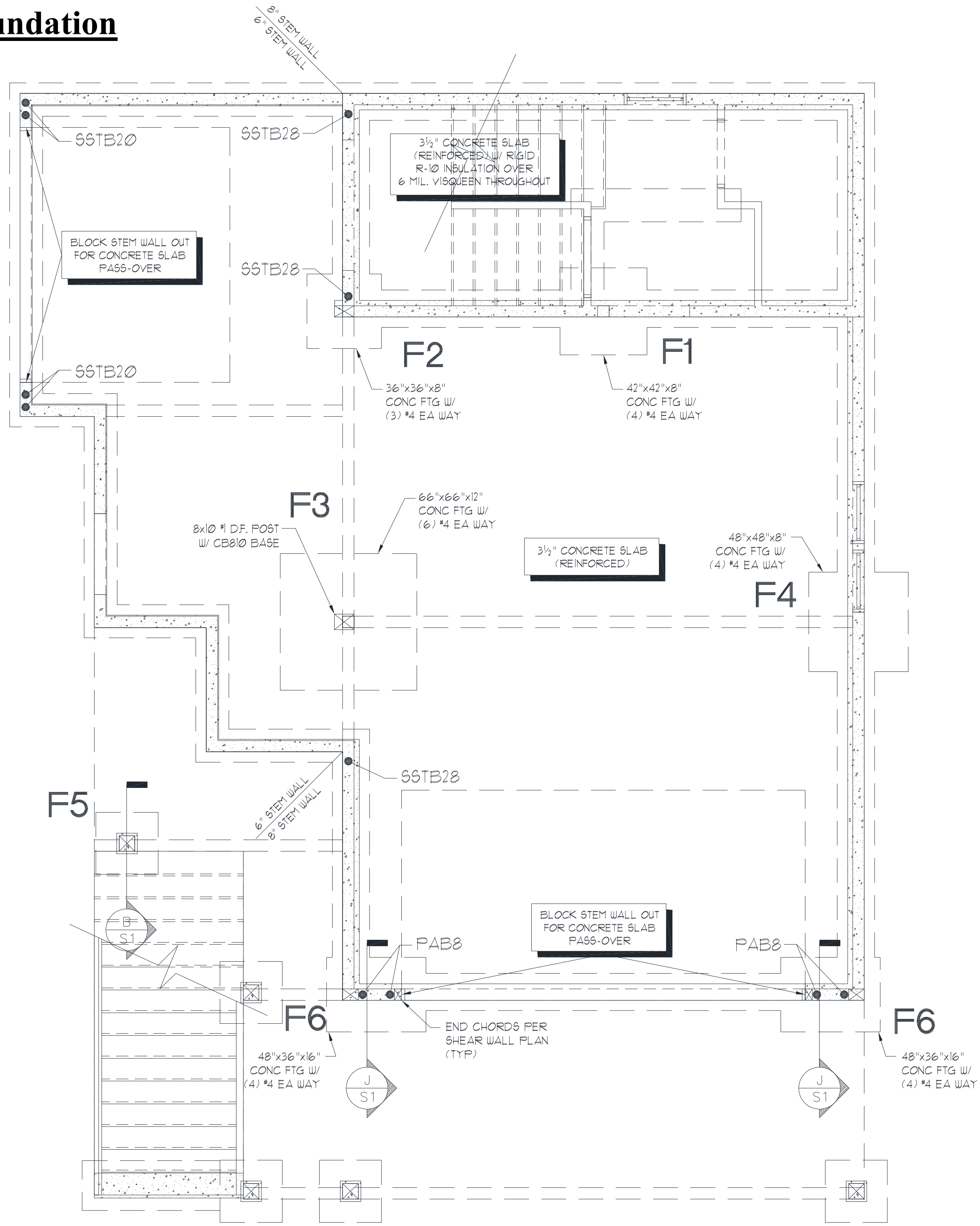
General Notes

- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports (1¾" minimum end bearing).
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **Spans shall be reduced 6" when floor panels are nailed only.**
- Spans generated from iLevel® software may exceed the spans shown in these tables because software reflects actual design conditions.
- For loading conditions not shown, refer to software or to the load table on page 5.

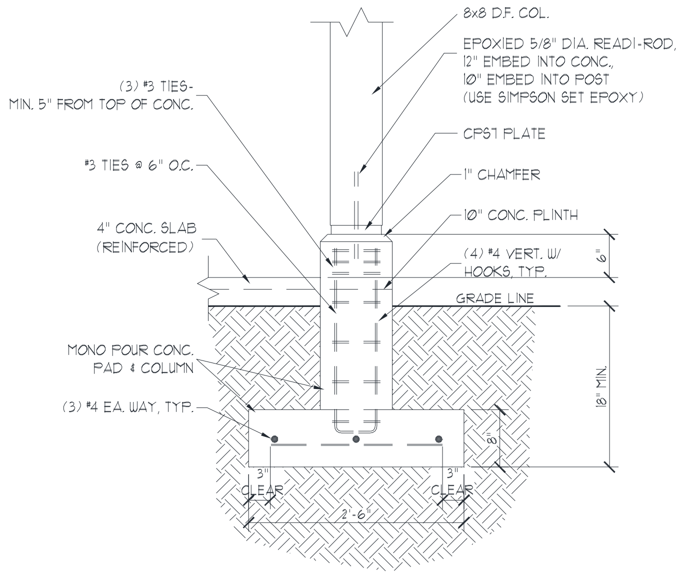
Therefore use:

11 7/8" TJI 210 floor joists @ 16" o.c. typical unless noted.

Foundation

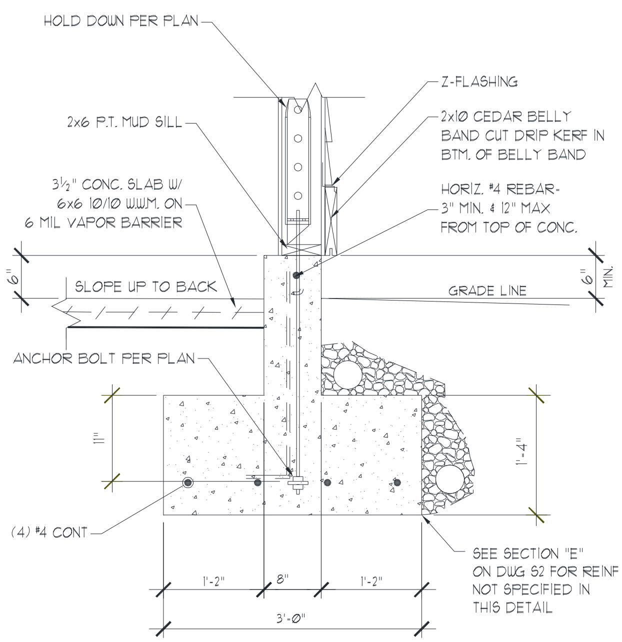


FOUNDATION PLAN



CONC. PLINTH & PAD DETAIL

B



GARAGE FTG. DETAIL

J

Typical Bearing Point Footing

P = 9,500 lbs
b = Pad Width = 1.33 ft
L = Pad Length = 4.5 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{9500}{(1.33)(4.5)}$ = 1,587 psf

Therefore the 16" wide footing is acceptable for loads under 9,500 lbs.

F1 - Bearing Point Footing

P = From R01+M03 = 15800+5340 = 21,140 lbs
b = Pad Width = 3.5 ft
L = Pad Length = 3.5 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{21{,}140}{(3.5)(3.5)}$ = 1,726 psf

Use 42"x42"x8" footing w/ (4) #4 each way.

F2 - Bearing Point Footing

P = From M06 = 15,000 lbs
b = Pad Width = 3 ft
L = Pad Length = 3 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{15000}{(3)(3)}$ = 1,667 psf

Use 36"x36"x8" footing w/ (3) #4 each way.

F3 - Bearing Point Footing

P = From M06 = 52,100 lbs
b = Pad Width = 5.5 ft
L = Pad Length = 5.5 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{52100}{(5.5)(5.5)}$ = 1,722 psf

Use 66"x66"x12" footing w/ (6) #4 each way.

F4 - Bearing Point Footing

P = From S01+M04 = 20,200+8430 = 28,630 lbs
b = Pad Width = 4 ft
L = Pad Length = 4 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{28630}{(4)(4)}$ = 1,789 psf

Use 48"x48"x8" footing w/ (4) #4 each way.

F5 - Bearing Point Footing

P = From R03+M10 = 7510+1802 = 9,312 lbs
b = Pad Width = 2.5 ft
L = Pad Length = 2.5 ft
σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{9312}{(2.5)(2.5)}$ = 1,490 psf

Use 30"x30"x8" footing w/ (3) #4 each way.

F6 - Bearing Point Footing

P = From S03+M07 = 11000+8362 = 19,362 lbs

b = Pad Width = 4 ft

L = Pad Length = 3 ft

σ = Soil Bearing Pressure = $\frac{P}{b\,L} = \frac{19362}{(4)(3)}$ = 1,614 psf

Use 48"x36"x16" footing w/ (4) #4 each way.