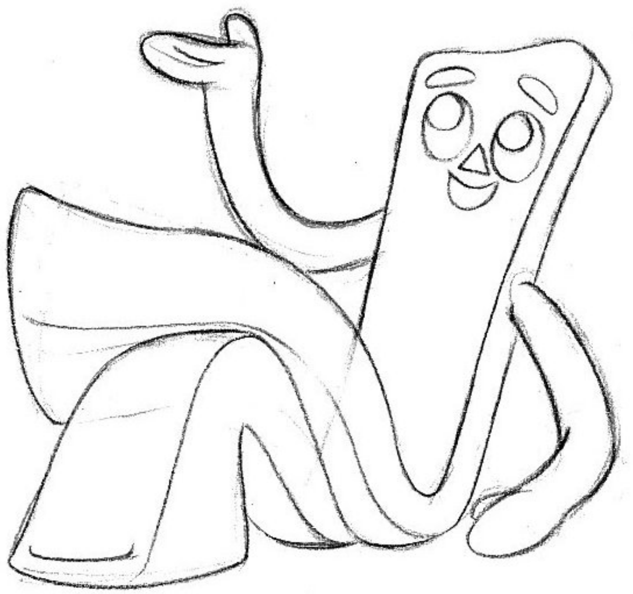
Accessory Dwelling Unit

1125 Easy St. Morgan Hill



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LOAD TAKEOFF

FLOOR LOADS					
Load Takeoff (psf)					
Description	Decking	Beams	Girders	Columns	Seismic
Finish	4	4	4	4	4
2x Decking	4.5	4.5	4.5	4.5	4.5
Plywood	1.5	1.5	1.5	1.5	1.5
Mep	2	2	2	2	2
Misc.	5	5	5	5	5
Beams		3.5	3.5	3.5	3.5
Girders			2	2	2
Columns				5.5	5.5
Walls					15
Total	17	20.5	22.5	28	43
Use	17	21	23	28	43
Const LL	20				
Attic LL	40				

ROOF LOADS					
Load Takeoff (psf)					
Sloped Roof	Rise	Run	Factor		
Great Room	6 1/3	12	1.13073		
Garage	7 3/8	12	1.17376		
Description	Decking	Beams	Girders	Columns	Seismic
Finish (Solar Shingles)	13	13	13	13	13
2x Decking	4.5	4.5	4.5	4.5	4.5
Plywood (2 layers)	1.5	1.5	1.5	1.5	1.5
Rigid Insulation (5")	1.25	1.25	1.25	1.25	1.25
Mep	2	2	2	2	2
Misc.	5	5	5	5	5
Beams		3.5	3.5	3.5	3.5
Girders			2	2	2
Columns				5.5	5.5
Walls					15
Total	31.98496	36.09312	38.44064	44.89632	62.50272
Use	32	37	39	45	63
Const LL	20				
Roof LL	20				

WALL LOADS					
Load Takeoff (psf)					
Description	Decking	Beams	Girders	Columns	Seismic
Wood Siding	2.5	2.5	2.5	2.5	2.5
1/2" Sheetrock	2.2	2.2	2.2	2.2	2.2
2x Framing	2	2	2	2	2
Plywood	1.5	1.5	1.5	1.5	1.5
Batt Insulation	1	1	1	1	1
Mep	2	2	2	2	2
Misc.	5	5	5	5	5
Total	16.2	16.2	16.2	16.2	16.2
Use	17	17	17	17	17

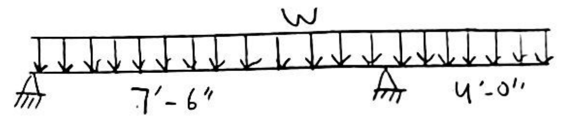
Gravity System Design (ASD)

Roof Framing

Typ. Rafter:

- Will use worst case for typical rafters
- Must check worst case for negative and positive bending

Worst Case Negative Bending:
(Horizontal Projection)



From Roof Load Takeoff, (Adjusted for the roof slope)

$DL = 37\ psf$
$LL = 20\ psf$ (Reducable)

LL Reduction:

$L_r = L_o R_1 R_2$

$A_T = W_{Trib}(L) = 4\,ft(11.5\,ft) = 46\,ft^2 \leq 200\,ft^2$

$\rightarrow R_1 = 1$

Roof Slope = $6\frac{1}{3}''$ per $12'' \rightarrow F = 6\frac{1}{3}''$

$R_2 = 1.2 - 0.05(6\frac{1}{3}'') = 0.88$

$L_r = 20\,psf(0.88)(1.0) = \underline{17.6\,psf}$

Load Combinations

1. $D = 37\,psf$
3. $D + L_r = 37 + 17.6 = 54.6\,psf \leftarrow$ Governs

$W = (D+L_r)\,\text{Trib Width} = (54.6\,psf)(4\,ft) = 218.4\,plf$

From Steel Manual Table 3-23, Case 24,

$$M_1 = \frac{W}{8L^2}(L+a)^2(L-a)^2 = \frac{218.4\,plf}{8(7.5\,ft)^2}(7.5+4)^2(7.5-4)^2 = \underline{786.27\,lbft}$$

$$M_2 = \frac{Wa^2}{2} = -\frac{218.4\,plf(4\,ft)^2}{2} = \underline{-1747.2\,lbft}$$

$\rightarrow$

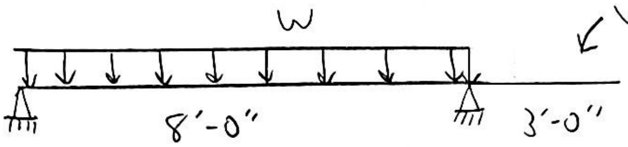
$$V_1 = \frac{W}{2L}(L^2 - a^2) = \frac{218.4\,plf}{2(7.5ft)}(7.5^2 - 4^2) = 586.04\,lbs$$

$$V_2 = Wa = 218.4\,plf(4ft) = 873.6\,lbs$$

$$V_3 = \frac{W}{2L}(L^2 + a^2) = \frac{218.4\,plf}{2(7.5ft)}(7.5^2 + 4^2) = 1051.96\,lbs$$

$\underline{V_{max} = 1051.96\,lbs}$

Worst Case Positive Bending:
(Horizontal Projection)



Worst Case would not have any Load on the Cantilever

- Same DL and L_r as Worst Case Negative Bending,

  $DL = 37psf$
  $L_r = 17.6psf$

- Governing Load Combo,

$$D + L_r = 37 + 17.6 = 54.6psf$$

$$W = (D + L_r)\,\text{Trib. Width} = 54.6psf(4ft) = 218.4plf$$

$$M_{max} = \frac{wL^2}{8} = \frac{218.4plf(8^2)}{8} = \underline{1747.2\,lbft}$$

$$V_{max} = \frac{wL}{2} = \frac{218.4plf(8)}{2} = \underline{873.6\,lb}$$

<u>Required I (Deflection Check)</u>

From IBC, $L/180$ for D+L and $L/240$ for L

- Worst Case $L = 11.5ft$

  $W_{D+L} = 218.4plf$
  $W_L = 17.6psf(4ft) = 70.4plf$

- Will be Conservative and use $\Delta = \frac{5wL^4}{384EI}$ for Deflection

$\longrightarrow$

$L/180 = \frac{11.5ft \times 12}{180} = 0.76''$

$L/240 = \frac{11.5ft \times 12}{240} = 0.575''$

using Glulams 24F-V8 DF/DF
$\rightarrow E = 1.8\times10^6 psi$

$$\Delta_{D+L} = \frac{5\,(218.4\,plf \times \frac{1}{12})(11.5\times12)^4}{384\,(1.8\times10^6)\,I} = 0.76''$$

$\rightarrow I_{req} = 62.83\,in^4 \leftarrow$ Governs

$$\Delta_L = \frac{5(70.4\,plf \times \frac{1}{12})(11.5\times12)^4}{384\,(1.8\times10^6)\,I} = 0.575''$$

$\rightarrow I_{req} = 26.77\,in^4$

- Checked Case of only Cantilever Load deflection and it was not close to governing

$I_{req} = 62.83\,in^4$

From Table 1C in the NDS,

Try $5\frac{1}{8}'' \times 6''$

Check Bending Strength

$F_b = 2400\,psi$

Adjustment Factors:

$C_M = 1.0$ $\quad C_c = 1.0$

$C_t = 1.0$ $\quad C_I = 1.0$

$C_{fu} = 1.0$ $\quad C_V = \left(\frac{21}{11.5}\right)^{1/10}\left(\frac{12}{6}\right)^{1/10}\left(\frac{5.125}{5\,1/8}\right)^{1/10} = 0.99 \approx 1.0$

$C_D = 1.25$

C_L

- Fully braced for positive bending, but not Negative bending

$$l_u = 11.5 - L\left(1 - \frac{a^2}{L^2}\right) = 11.5 - 7.5\left(1 - \frac{4^2}{7.5^2}\right) = 6.13\,ft$$

↑ From Steel Manual Table 3-23

$$l_u/d = \frac{6.13\times12}{6} = 12.26 \rightarrow l_e = 1.63(6.13\times12) + 3(6) = 137.9$$

$$R_B = \sqrt{\frac{l_e d}{b^2}} = \sqrt{\frac{137.9(6)}{(5.125)^2}} = 5.6125 \rightarrow F_{bE} = \frac{1.2(0.95\times10^6)}{5.61^2} = 36222.56$$

$$F_{BE}/F_b^* = 36222.56/2400 = 15.09$$

$$C_L = \frac{1+15.09}{1.9} - \sqrt{\left(\frac{1+15.09}{1.9}\right)^2 - \frac{15.09}{0.95}} \rightarrow C_L = 8.47 - 7.47 = 1.0$$

$F_b' = 2400 \times 1.25 = 3000\ psi$

$f_{b+} = \frac{M_+}{S} = \frac{1747.2\ lbft \times 12}{30.75\ in^3} = 681.83\ psi$

$f_{b-} = \frac{M_-}{S} = \frac{-1747.2\ lbft \times 12}{30.75\ in^3} = 681.83\ psi$

$F_b' = 3000\ psi > f_b = 681.83\ psi$ ✓

$DCR = \frac{681.83}{3000} = \underline{0.227}$

<u>Check Shear Strength</u>

$V_{max} = 1051.96\ lbs$

Adjustment Factors:

$C_D = 1.25$ $C_{vr} = 1.0$

$C_M = 1.0$

$C_t = 1.0$

$F_v = 265\ psi$

$F_v' = 265 \times 1.25 = 331.25\ psi$

$f_v = \frac{1.5V}{A} = \frac{1.5(1051.96\ lbs)}{30.75} = 51.32\ psi$

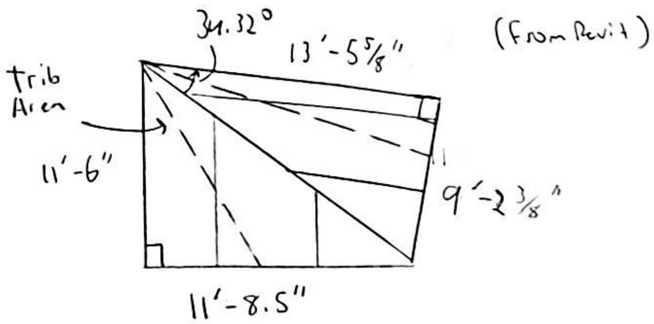
$F_v' = 265\ psi > f_v = 51.32\ psi$ ✓

$DCR = \frac{51.32\ psi}{331.25\ psi} = \underline{0.155}$

Use GLB 5⅛" x 6" or bigger
24F-V8

Valley Rafter Design

Valley 1:

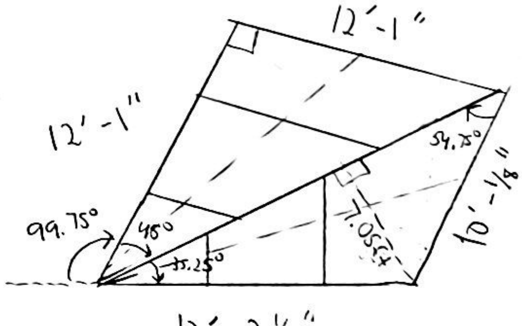


Trib Area = ½ total Area

$$\rightarrow \left[\tfrac{1}{2}(11.5ft \times 11.708ft) + \tfrac{1}{2}(13.47ft \times 9.2ft)\right]\tfrac{1}{2}$$

Trib Area = $\underline{74.5\ ft^2}$

Valley 2:



Trib Area = ½ total Area

$$\rightarrow \tfrac{1}{2}\left[\tfrac{1}{2}(12'\text{-}1")^2 + \tfrac{1}{2}(12'\text{-}2.5" \times 7.05') + \tfrac{1}{2}(10'\text{-}\tfrac{1}{8}" \times 7.05')\right]$$

Trib AREA = $\underline{75.66\ ft^2}$ ← Governs

Loads:

$D_L = 39$ psf

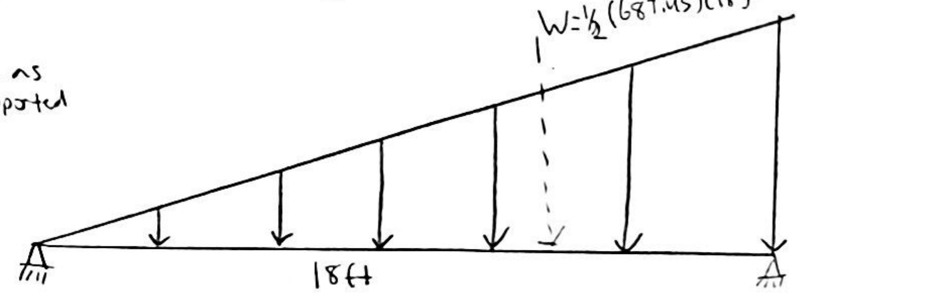
$L_r = 17.6$ psf (Same Lr since A_T is still less than 200)

$D+L = 39 + 17.6 = 56.6$ psf

$W_{max} = (D + L_r)(\text{Trib. width @ End})$

$= 56.6\,psf\left(\frac{12'\text{-}1'' + 12'\text{-}2.5''}{2}\right) = \underline{687.45\ plf}$

- Idealized as Simply supported



$M_{max} = 0.128\,WL = 0.128(6187.05\ lbs)(18\,ft) = \underline{14254.96\ lbft}$

$V_{max} = \frac{2W}{3} = \frac{2}{3}(6187.05\,lb) = \underline{4124.7\ lbs}$

<u>Required I (Deflection Check)</u>

$L/180 = \frac{18ft \times 12}{180} = 1.2''$

$\Delta_{max} = 0.013\frac{WL^3}{EI} = 0.013\frac{(6187.05\,lbs)(18\,ft \times 12)^3}{(1.8\times10^6 psi)(I)} = 1.2$

$\underline{I_{req'd} = 375.26\ in^4}$

From Table 1C,

<u>Try 5 1/8" x 10 1/2" GLB</u>

<u>Bending Check</u>

$F_b = 2400\,psi$

Adjustment Factors:

$C_D = 1.25$

$C_L = 1.0$ (fully braced from Sheathing)

$C_v = \left(\frac{21}{18}\right)^{1/10}\left(\frac{12}{10.5}\right)^{1/10}\left(\frac{5.125}{5.125}\right)^{1/10} = 1.029 \rightarrow 1.0$

$F_b' = 3000\ psi$

$f_b = \frac{M}{S} = \frac{14254.96\,lbft \times 12}{94.17\,in^3} = 1816.5\,psi$

$F_b' = 3000\,psi > f_b = 1816.5\,psi$ ✓

$DCR = \frac{1816.5}{3000} = \underline{0.606}$

Shear Check

$V_{max} = 4124.7 lbs$

$F_v' = 331.25 psi$

$f_v = \frac{1.5V}{A} = \frac{1.5(4124.7 lbs)}{53.81 in^2} = 114.98 psi$

$F'_v = 331.25 psi > f_v = 114.98 psi$ ✓

$DCR = \frac{114.98}{331.25} = \underline{0.347}$

$\boxed{\text{Use } 5\tfrac{1}{8}" \times 10\tfrac{1}{2}" \text{ GLB 24F-V8 DF/DF or bigger}}$

Ridge Beam Design

Ridge Beam Over Garage

$L = 18'\text{-}10''$

$\text{Trib width} = \frac{11.5'}{2} + \frac{10.5'}{2} = 11\text{ ft}$

$D_L = 39\text{ psf}$

$L_r = 17.6\text{ psf}$

$W = (D + L_r)\text{Trib width} = (39+17.6)(11\text{ft}) = 622.6\text{ plf}$

Max Loads:

$$M_u = \frac{wL^2}{8} = \frac{(622.6\text{plf})(18.833\text{ft})^2}{8} = \underline{27{,}604.1\text{ lbft}}$$

$$V_u = \frac{wL}{2} = \frac{(622.6\text{ plf})(18.833\text{ft})}{2} = \underline{5862.82\text{ lbs}}$$

Required I

$$L/180 = \frac{18\times12+10}{180} = 1.25''$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(622.6\text{ plf}\times 1/12)(18\times12+10)^4}{384(1.8\times10^6)\,I} = 1.25''$$

$\underline{I_{req'd} = 783.28\text{ in}^4}$

$\underline{\text{Try } 6\tfrac{3}{4}''\times12''\text{ GLB}}$

Bending Check

$F_b = 2400\text{ psi}$

$$C_v = \left(\frac{21}{18.83}\right)^{1/10}\left(\frac{12}{12}\right)^{1/10}\left(\frac{5.125}{6.75}\right)^{1/10} = 0.98 \qquad C_D = 1.25$$

C_L for bracing every 4ft from rafters = 1.0

$F'_b = 2400(0.98)(1.25) = 2940$

$$f_b = \frac{M}{S} = \frac{27{,}604.1\text{ lbft}\times12}{162\text{ in}^3} = 2044.75\text{ psi}$$

$F'_b = 2940\text{ psi} > f_b = 2044.75\text{ psi}$ ✓

$$DCR = \frac{2044.75}{2940} = \underline{0.695}$$

Shear Check

$V_{max} = 5862.82 \text{ lbs}$

$F_v' = 331.25 \text{ psi}$

$$f_v = \frac{1.5V}{A} = \frac{1.5(5862.82)}{81 \text{ in}^2} = 108.57 \text{ psi}$$

$F_v' = 331.25 \text{ psi} > f_v = 108.57 \text{ psi}$ ✓

$$DCR = \frac{108.57}{331.25} = \underline{0.328}$$

Use GLB 24F-V4 $6\frac{3}{4}" \times 12"$ or bigger

Ridge Beam Over Great Room

* Split up into 2 simply supported beams, seam over column that drops down to the Loft

Longer Span = $16' 9\frac{3}{8}" \rightarrow 17'$

Trib Width = $\frac{18' + 6'}{2} = 12'$

$D_L = 39 \text{ psf}$

$L_r = 17.6 \text{ psf}$

$w = (D + L_r)\text{Trib width} = (39 + 17.6)(12 \text{ ft}) = 679.2 \text{ plf}$

Max Loads:

$$M_u = \frac{wL^2}{8} = \frac{679.2 \text{ plf}(17 \text{ ft})^2}{8} = \underline{24{,}536.1 \text{ lbft}}$$

$$V_u = \frac{wL}{2} = \frac{679.2 \text{ plf}(17 \text{ ft})}{2} = \underline{5773.2 \text{ lbs}}$$

Required I

$$L/180 = \frac{17 \times 12}{180} = 1.1\overline{3}"$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(679.2 \text{ plf} \cdot \frac{1}{12})(17 \times 12)^4}{384(1.8 \times 10^6) I} = 1.1\overline{3}"$$

$$\underline{I_{req'd} = 625.67 \text{ in}^4}$$

Try $6\frac{3}{4}" \times 10\frac{1}{2}"$ GLB

Bending Check

$f_b = 2400\,psi \qquad C_D = 1.25$

$C_V = \left(\frac{21}{17}\right)^{1/10}\left(\frac{12}{10.5}\right)^{1/10}\left(\frac{5.125}{6.75}\right)^{1/10} = 1.0$

C_L for bracing every 4ft from Rafters = 1.0

$F'_b = 3000\,psi$

$f_b = \frac{M}{S} = \frac{24{,}536.1\ lbft \times 12}{124.0\,in^3} = 2374.46\,psi$

$F'_b = 3000\,psi > f_b = 2374.46\,psi$ ✓

$DCR = \frac{2374.46}{3000} = \underline{0.791}$

Shear Check

$F_V' = 265 \times \overset{C_D}{1.25} = 331.25$

$f_v = \frac{1.5V}{A} = \frac{1.5(5773.2)}{70.88} = 122.18\,psi$

$V_{max} = 5773.2$

$F'_v = 265 > f_v = 122.18\,psi$ ✓

$DCR = \frac{122.18}{331.25} = \underline{0.369}$

$\boxed{\text{Use } 6^3/_4" \times 10^1/_2" \text{ GLB 24F-V4 or bigger}}$

* Use Same Size for Shorter span

Ridge Beam Over the Loft

$L = 28'\text{-}10\tfrac{1}{2}'' \rightarrow 29'$

$\text{Trib width} = \frac{11.5'}{2} + \frac{10.5'}{2} = 11\text{ ft}$

$DL = 39\text{ psf}$

$L_r = 17.6\text{ psf}$

$W = (D_L + L_r)\,\text{Tribwidth} = (39 + 17.6)(11\text{ft}) = 622.6\text{ plf}$

* Point Load At the Valleys and other roof ridge beam

$P_{valley} = \frac{2W}{3} = \frac{2(6187.05)}{3} = 4124.7\text{ lbs}$ (from Valley Rafter design)

Short Ridge over Great Room

$w = 679.2\text{ plf}$ (from ridge over great room design)

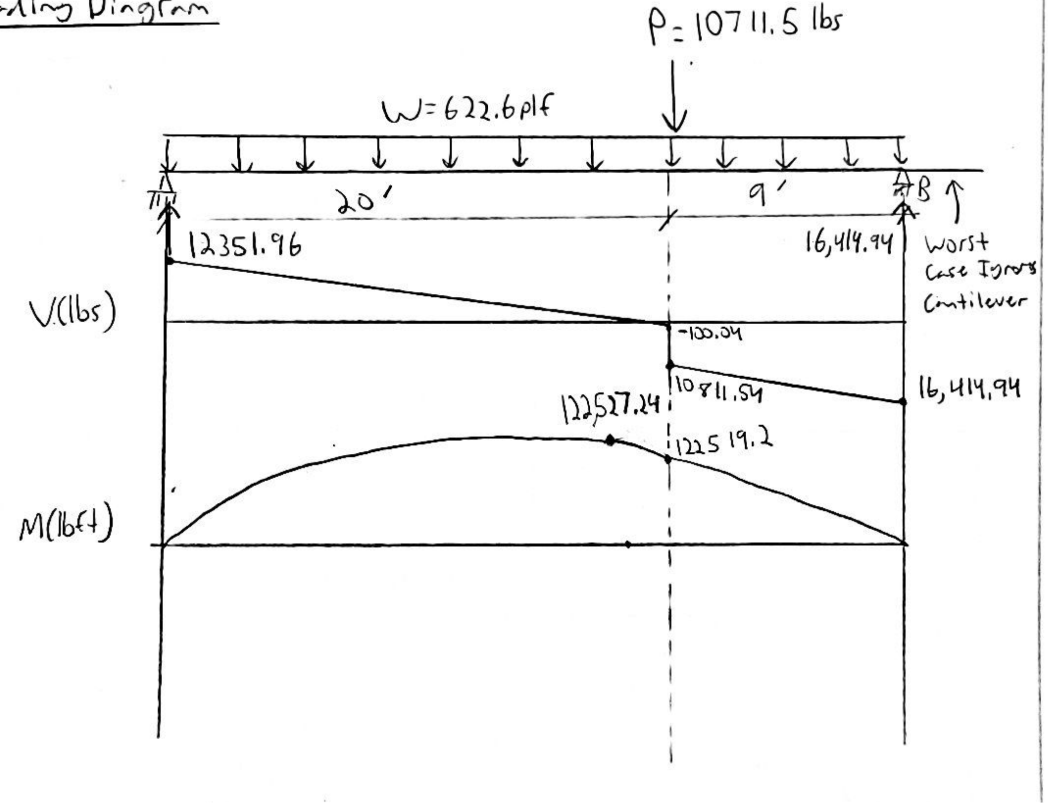
7'-3"

$P_{ridge} = \frac{wL}{2} = \frac{679.2\text{plf}(7.25\text{ft})}{2} = 2462.1\text{ lbs}$

$P = 2(P_{valley}) + P_{ridge} = 2(4124.7) + 2462.1$

$P = 10711.5\text{ lbs}$

Loading Diagram



Required I

$$P_{50\%} = \left(\tfrac{39}{2}+17.6\right)\left(\frac{12'\text{-}1'' + 12'\text{-}2\tfrac{1}{2}''}{2}\right)\left(\frac{18'}{2}\right)\left(\tfrac{2}{3}\right)(2) + 2462.1 = 7869.425 \text{ lbs}$$

$$W_{50\%} = \left(\tfrac{39}{2}+17.6\right)(11\text{ft}) = 408.1 \text{ plf}$$

$$\Delta = \frac{5wL^4}{384EI} + \frac{P a b (a+2b)\sqrt{3a(a+2b)}}{27EIL}$$

$a = 20\text{ft} = 240\text{ in}$
$b = 9\text{ft} = 108\text{ in}$

$$= \frac{5(408.1\times\frac{1}{12})(29\times12)^4}{384(1.8\times10^6)\, I} + \frac{7869.43(240)(108)(240+2(108))\sqrt{3(240)(240+2(108))}}{27(1.8\times10^6)(348)\, I}$$

$$= \frac{3608.02}{I} + \frac{3151.2}{I} = \frac{6759.22}{I} = L/180 = \frac{29\times12}{180} = 1.93''$$

$$\underline{I_{req'd} = 3502.19 \text{ in}^4}$$

Try $6\tfrac{3}{4}'' \times 19\tfrac{1}{2}''$ GLB

Bending Check

$F_b = 2400$ psi

$C_D = 1.25$

$C_V = \left(\frac{21}{29}\right)^{1/10}\left(\frac{12}{19.5}\right)^{1/10}\left(\frac{5.125}{6.75}\right)^{1/10} = 0.90$ ← Governs

C_L for bracing every 4 ft from rafters $= 0.99$

$F_b' = 2400(0.90)(1.25) = 2700$ psi

$f_b = \frac{M}{S} = \frac{122{,}527.24\times12}{427.8} = 3436.95$ psi N.G.

→ Try $6\tfrac{3}{4}'' \times 22\tfrac{1}{2}''$ GLB

$C_V = \left(\frac{21}{29}\right)^{1/10}\left(\frac{12}{22.5}\right)^{1/10}\left(\frac{5.125}{6.75}\right)^{1/10} = 0.88$ ← Governs

$C_L = 1.0$

$F_b' = 2400(0.88)(1.25) = 2640$ psi

$f_b = \frac{122{,}527.24\times12}{569.5} = 2581.78$

$F_b' = 2640 \text{ psi} > f_b = 2581.78$ ✓

$DCR = \frac{2581.78}{2640} = \underline{0.977}$

Shear Check

$F_v' = 331.25\,psi$

$V_{max} = 16{,}414.94\ lbs$

$$f_v = \frac{1.5V}{A} = \frac{1.5(16414.94)}{196.9} = 125.1\,psi$$

$F_v' = 331.25\,psi > f_v = 125.1\,psi$ ✓

$$DCR = \frac{125.1}{331.25} = \underline{0.378}$$

$\boxed{\text{Use } 6\tfrac{3}{4}'' \times 22\tfrac{1}{2}'' \text{ GLB 24F-V4 or Bigger}}$

Worst Case King Post

- Post supporting Loft Ridge Beam

$P = 16{,}414.94 \text{ lbs}$

$L = 3.5 \text{ ft}$

Use Sawn Lumber for king Posts,

$F_c = 1350$ (DF-L NO.2)

Adjustment Factors:

$C_D = 1.25$

$C_m = 1.0$

$C_t = 1.0$

Estimate Size:

$$f_c = \frac{P}{A} = \frac{16414.94 \text{ lbs}}{A} = 1350(1.25)$$

$$A_{req} = 9.73 \text{ in}^2$$

Try 4x4

C_P

$$l_e = k_e(l) = 1.0(3.5 \times 12) = 42$$

$$l_e/d = 42/3.5 = 12 < 50 \checkmark$$

$$F_{cE} = \frac{0.822(0.95 \times 10^6)}{12^2} = 5422.9$$

$$F_{cE}/F_c^* = \frac{5422.9}{(1350 \times 1.25)} = 3.21$$

$$C_P = \frac{1+3.21}{2(0.9)} - \sqrt{\left[\frac{1+3.21}{2(0.9)}\right]^2 - \frac{3.21}{0.9}} = 2.34 - 1.38 = 0.93$$

$$F_c' = 1350(0.93)(1.25) = 1569.38 \text{ psi}$$

$$f_c = \frac{P}{A} = \frac{16414.94 \text{ lbs}}{12.25 \text{ in}^2} = 1339.99 \text{ psi} \checkmark$$

$$DCR = \frac{1339.99}{1569.38} = \underline{0.854}$$

$\boxed{\text{Use 4x4 DF-L NO.2 for King Posts}}$

Worst Case Header

- End of the Ridge over the loft where the exterior window is

Loading

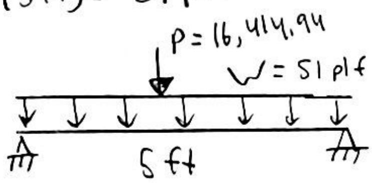
$P_{ridge} = 16{,}414.94 \text{ lbs}$

Wall weight:

$D_L = 17 \text{ psf}$

Height above header = 3 ft

$W = 17 \text{ psf} (3 \text{ ft}) = 51 \text{ plf}$



$$M_u = \frac{PL}{4} + \frac{WL^2}{8} = \frac{16{,}414.94(5\text{ft})}{4} + \frac{(51 \text{ plf})(5\text{ft})^2}{8} = \underline{20{,}678.05 \text{ lbft}}$$

$$V_u = \frac{P}{2} + \frac{WL}{2} = \frac{16{,}414.94}{2} + \frac{51(5)}{2} = \underline{8334.97 \text{ lbs}}$$

Required A

$F_v = 265 \text{ psi}$

$$f_v = \frac{1.5V}{A} = \frac{1.5(8334.97)}{A} = 265 \rightarrow A_{req} = 47.18 \text{ in}^2$$

Try $\underline{5\tfrac{1}{8}" \times 10\tfrac{1}{2}"}$

Check Bending

$F_b = 2400 \text{ psi}$

Adjustment Factors: $C_D = 1.25$, $C_V = \left(\frac{21}{5}\right)^{1/10}\left(\frac{12}{10.5}\right)^{1/10}\left(\frac{5.125}{5.125}\right)^{1/10} = 1.0$

C_L (unbraced) = 0.99

$F_b' = 2400(0.99)(1.25) = 2970 \text{ psi}$

$$f_b = \frac{M}{S} = \frac{20{,}678.05 \text{ lbft} \times 12}{94.17 \text{ in}^3} = 2634.98 \checkmark$$

$$DCR = \frac{2634.98}{2970} = \underline{0.887}$$

Check Deflection

$$L/240 = 5\times12/240 = 0.25''$$

$$\Delta = \frac{5wL^4}{384EI} + \frac{PL^3}{48EI} = \frac{5(51\times\frac{1}{12})(5\times12)^4}{384(1.8\times10^6)(738)} + \frac{16414.94(5\times12)^3}{48(1.8\times10^6)(738)} = 0.056$$

$$\Delta = 0.056'' < 0.25'' \checkmark$$

$$\boxed{\text{Use } 5\tfrac{1}{8}'' \times 10\tfrac{1}{2}'' \text{ GLB 24F-V4}}$$

Typical Loft Joist

$L = 15\,ft$

$Trib\,width = 4\,ft$

$DL = 21\,psf$

$LL = 40\,psf$

$$W = (21\,psf + 40\,psf)\,4\,ft = 244\,plf$$

$$M_u = \frac{wL^2}{8} = \frac{244\,plf\,(15\,ft)^2}{8} = 6862.5\ lbft$$

$$V_u = \frac{wL}{2} = \frac{244\,plf\,(15\,ft)}{2} = 1830\ lbs$$

Required I

* Governed by LL Deflection so check →

$$L/360 = \frac{15 \times 12}{360} = 0.5''$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5\left(244\,plf \times \frac{1}{12}\right)(15 \times 12)^4}{384\,(1.8 \times 10^6)\ I} = 0.5''$$

$$I_{req'd} = 308.81\ in^4$$

Try $5\frac{1}{8}'' \times 9''$ GLB

Bending Check

$F_b = 2400\ psi$

Adjustment Factors: $C_D = 1.0$, $C_V = 1.0$, C_L (fully braced) $= 1.0$

$F'_b = 2400\ psi$

$$f_b = \frac{M}{S} = \frac{6862.5 \times 12}{69.19} = 1190.2\ psi$$

$F_b' = 2400 > f_b = 1190.2\ psi$ ✓

$$DCR = \frac{1190.2}{2400} = \underline{0.496}$$

Shear Check

$F'_v = 265\ psi$

$$f_v = \frac{1.5V}{A} = \frac{1.5\,(1830)}{46.13} = 59.5\ psi < F'_v = 265\ psi\ ✓$$

$$DCR = \frac{59.5}{265} = \underline{0.225}$$

$$\boxed{\text{Use } 5\tfrac{1}{8}'' \times 9'' \text{ GLB 24F-V4}}$$

Worst Case Roof Girder

$L = 21' \, 6\frac{3}{4}''$

$\text{Trib width} = 3' + \frac{9'}{2} = 7.5 \text{ ft}$

$A_T = (21.5625')(7.5') = 161.72 \text{ ft}^2 < 200 \text{ ft}^2$ (Same L_L reduction)

$D_L = 39 \text{ psf}$

$L_r = 17.6 \text{ psf}$

$W_u = (39 + 17.6 \text{ psf})(7.5 \text{ ft}) = 424.5 \text{ plf}$

$$M_u = \frac{wL^2}{8} = \frac{424.5 \text{ plf}(21.5625 \text{ ft})^2}{8} = \underline{24,670.95 \text{ lbft}}$$

$$V_u = \frac{wL}{2} = \frac{424.5 \text{ plf}(21.5625 \text{ ft})}{2} = \underline{4576.64 \text{ lbs}}$$

Required I

$$L/180 = \frac{21.5625 \times 12}{180} = 1.4375''$$

$$\Delta = \frac{5wL^4}{384EI} = \frac{5\left(424.5 \text{ plf} \times \frac{1}{12}\right)(21.5625 \times 12)^4}{384(1.8 \times 10^6) I} = 1.4375''$$

$\underline{I_{req} = 797.95 \text{ in}^4}$

Try $5\frac{1}{8}'' \times 13\frac{1}{2}''$ GLB

Bending Check

$F_b = 2400 \text{ psi}$ Adjustment Factors: $C_D = 1.25$

$$C_V = \left(\frac{21}{21.5625}\right)^{1/10}\left(\frac{12}{13.5}\right)^{1/10}\left(\frac{5.125}{5.125}\right)^{1/10} = 0.98$$

C_L (braced every 4 ft) $= 1.0$

$F_b' = 2400(0.98)(1.25) = 2940 \text{ psi}$

$$f_b = \frac{M}{S} = \frac{24670.95 \times 12}{155.7} = 1901.42 < F_b' = 2940 \text{ psi} \checkmark$$

$$OCR = \frac{1901.42}{2940} = \underline{0.65}$$

Shear Check

$F_v' = 265 \times 1.25 = 331.25 \text{ psi}$

$$f_v = \frac{1.5V}{A} = \frac{1.5(4576.64)}{69.19 \text{ in}^2} = 99.22 \text{ psi} < F_v' = 331.25 \text{ psi} \checkmark$$

$$OCR = \frac{99.22}{331.25} = \underline{0.299}$$

$\boxed{\text{Use } 5\frac{1}{8}'' \times 13\frac{1}{2}'' \text{ GLB 24F-V4}}$

Worst Case Columns

Case 1: Column Carrying both Loft and Roof Loads

Roof Loads =

Trib. Area = $\frac{15.5 + 12.66}{2} \times \left(3 + \frac{7.5}{2}\right) = 95.06\ ft^2$

$D_L = 45\,psf$

$L_L = 17.6\,psf$

$P_{roof} = (45 + 17.6)(95.06) = 5950.76\,lbs$

Loft Loads

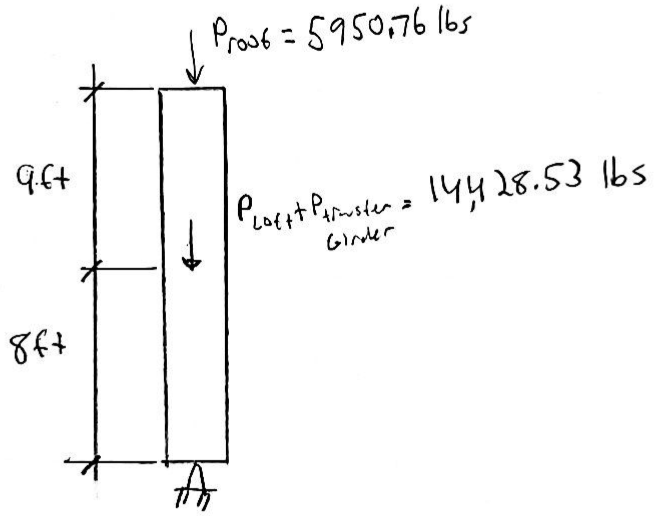
Trib Area = $\frac{12.66}{2} \times \left(\frac{15}{2}\right) = 47.48\ ft^2$

$D_L = 28\,psf$

$L_L = 40\,psf$

$P_{Loft} = (28 + 40)(47.48\,ft^2) = 3228.64\,lbs$

$P_{transfer\ Girder} = 11{,}199.89\,lbs$ (from transfer Girder Calcs)



$F_c = 1650\,psi$ (For 24F-V4 GLB)

Adjustment Factors =

$C_D = 1.0$

$C_m = 1.0$

$C_t = 1.0$

Estimate Size

$$f_c = \frac{P}{A} = \frac{5950.76 + 14{,}428.53}{A} = 1650\text{psi} \rightarrow A_{req} = 12.17\,\text{in}^2$$

Try $6\frac{3}{4}'' \times 7\frac{1}{2}''$ GLB

C_p

$$\ell_e = K_e(\ell) = 1.0(8\,ft \times 12) = 96''$$

$$\ell/d = \frac{96''}{6.75''} = 14.2 < 50 \checkmark$$

$$F_{cE} = \frac{0.822(0.95 \times 10^6)}{(14.2)^2} = 3872.74$$

$$\frac{F_{cE}}{F_c^*} = \frac{3872.74}{1650} = 2.35$$

$$C_p = \frac{1+(2.35)}{1.8} - \sqrt{\left(\frac{1+2.35}{1.8}\right)^2 - \frac{2.35}{0.9}} = 1.86 - 0.923$$

$$C_p = 0.937$$

$$F_c' = 1650\text{psi}(0.937) = 1546.05\text{ psi}$$

$$f_c = \frac{P}{A} = \frac{20377.29}{50.63} = 402.51\text{psi} \checkmark$$

$$DCR = \frac{402.51}{1546.05} = \underline{0.26}$$

$\boxed{\text{Use } 6\tfrac{3}{4}'' \times 7\tfrac{1}{2}'' \text{ GL 24F-V4}}$

Case 2: Column Carrying Garage and Loft Ridge Beam

Roof Loads

$$P = P_{\substack{Garage\\Ridge}} + P_{\substack{Loft\\Ridge}} = 5862.82 + 12351.96 = 18214.78\,lbs$$

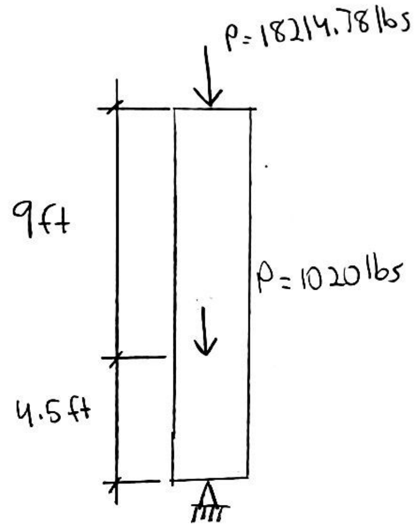
Floor Loads

$$\text{Trib Area} = \left(\frac{15ft}{2}\right)\left(\frac{4ft}{2}\right) = 15ft^2$$

$D_L = 28\,psf$

$L_L = 40\,psf$

$$P_{floor} = (28+40)(15ft^2) = 1020\,lbs$$



$F_c = 1650\,psi$ (for 24F-V4 GLB)

Adjustment Factors:

$C_D = 1.25$

$C_M = 1.0$

$C_t = 1.0$

Estimate Size:

$$f_c = \frac{P}{A} = \frac{19234.78}{A} = 1650 \times 1.25 \Rightarrow A_{req} = 9.32\,in^2$$

Try $5\frac{1}{8}" \times 6"$

CP

$$l_e = K_e l = (1.0)(9ft \times 12) = 108"$$

$$l_e/d = 108"/5.125 = 21.1 < 50 \checkmark$$

$$F_{cE} = \frac{0.822E'_{min}}{(l_e/d)^2} = \frac{0.822(0.95\times10^6)}{(21.1)^2} = 1758.47\text{ psi}$$

$$F_{cE}/F_c^* = \frac{1758.47}{(1650\times1.25)} = 0.853$$

$$C_p = \frac{1+0.853}{2(0.9)} - \sqrt{\left[\frac{1+0.853}{2(0.9)}\right]^2 - \frac{0.853}{0.9}} = 1.0294 - 0.334 = 0.695$$

$$F'_c = 1650(1.25)(0.695)$$
$$= 1433.44\text{ psi}$$

$$f_c = \frac{P}{A} = \frac{19234.78\text{ lbs}}{30.75\text{ in}^2} = 625.52\text{ psi} \checkmark$$

$$DCR = \frac{625.52}{1433.44} = \underline{0.436}$$

$$\boxed{\text{use } 5\tfrac{1}{8}'' \times 6'' \text{ GL 24F-V4}}$$

Transfer Girder Design

Wind Loads

Steps from ASCE 7-16 Table 30.3-1

Step 1: Risk Category = II

Step 2: Basid Wind Speed

$V = 93\ mph$

Step 3: Wind Load Parameters

$K_d = 0.85$ (Table 26.6-1)

Exposure Category = B

$K_{zt} = 1.0$

$K_e = 1.0$

Enclosure Classification: Enclosed

$GC_{pi} = \pm 0.18$

Step 4: Exposure Coefficient

$K_h = 0.59$ (Table 26.10-1)

Step 5: Velocity Pressure

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$$
$$= 0.00256(1.0)(1.0)(1.0)(0.85)(93mph)^2$$
$$= 18.82\ psf$$

Step 6: External Pressure Coefficient

Effective Wind Area $= (18ft \times 11.5ft) + (5.5ft \times 9ft) = 256.5\ ft^2$

$GC_p = \begin{matrix} +0.8 \\ -0.9 \end{matrix}$

Step 7: Wind Pressure

$$p = q_h(GC_p - GC_{pi})$$
$$= 18.82(0.8+0.18) = 18.44\ psf$$
$$= 18.82(-0.9-(0.18)) = -20.33\ psf$$

$$\boxed{p = 20.33\ psf}$$

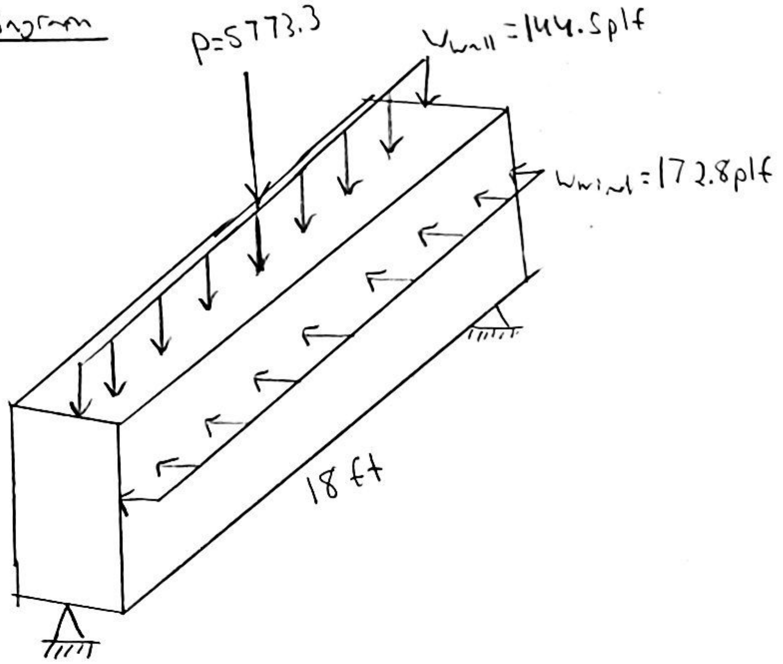
Trib Height of the Girder $= \frac{8.5\,ft}{2} + \frac{17-8.5\,ft}{2} = 8.5\,ft$

$W_{wind} = 20.33\,psf\,(8.5\,ft) = 172.8\ plf$

$P_{ridge} = 5773.3\ lbs$

$W_{wall} = 17\,psf\,(8.5\,psf) = 144.5\,plf$

Loading Diagram



Load Combos

1. D+Lr
2. D+0.75Lr + 0.75 (0.6W)
3. D + 0.6W

$$P_{0.75Lr} = \frac{[39+17.6(0.75)](12ft)(17ft)}{2} = 5324.4\ lbs$$

$$P_{DLr} = \frac{(39)(12ft)(17ft)}{2} = 3978\ lbs$$

Case 1

$$M_u = \frac{5773.3\,lbs\,(18\,ft)}{4} + \frac{144.5\,plf\,(18ft)^2}{8} = 31{,}832.1\ lbft$$

$$V_u = \frac{5773.3}{2} + \frac{144.5\,(18)}{2} = 4187.15\ lbs$$

Req'd I

$$L/240 = \frac{18\times12}{240} = 0.9''$$

$$\Delta = \frac{5wL^4}{384EI} + \frac{PL^3}{48EI} = \frac{5\left(144.5\times\frac{1}{12}\right)(18\times12)^4}{384(1.8\times10^6)\,I} + \frac{5773.3(18\times12)^3}{48(1.8\times10^6)\,I}$$

$$= \frac{189.61}{I} + \frac{673.4}{I} = 0.9''$$

$$I_{req'd} = 958.9\,in^4$$

Try 5 1/8" x 13 1/2" GLB

Check Bending

$F_b = 2400$ psi

Adjustment Factors:

$C_D = 1.25 \quad C_t = 1.0$

$C_m = 1.0 \quad C_L(\text{unbraced}) = 0.95$

$$C_V = \left(\frac{21}{18}\right)^{1/10}\left(\frac{12}{13.5}\right)^{1/10}\left(\frac{5.125}{5.125}\right)^{1/10} = 1.0$$

$$F_b' = 2400(1.25)(0.95) = 2850\text{ psi}$$

$$f_b = \frac{M}{S} = \frac{31{,}832.1\times12}{155.7} = 2453.34\text{ psi} ✓$$

$$DCR = \frac{2453.34}{2850} = \underline{0.86}$$

Check Shear

$$F_v' = 265\text{psi}(1.25) = 331.25\text{ psi}$$

$$f_v = \frac{1.5V}{A} = \frac{1.5(4187.15)}{69.19} = 90.78 ✓$$

$$DCR = \frac{90.78}{331.25} = \underline{0.274}$$

→ 5 1/8" x 13 1/2" GLB 24F-V4

Case 2

$$M_{ux} = \frac{5324.4(18)}{4} + \frac{144.5(18)^2}{8} = 29{,}812.05 \text{ lbft}$$

$$M_{uy} = \frac{0.75(0.6(172.8 \text{ plf}))(18 \text{ ft})^2}{8} = 3149.28 \text{ lbft}$$

$$V_u = \frac{5324.4}{2} + \frac{144.5(18)}{2} = 3962.7 \text{ lbs}$$

- Shear and Moment about Strong Axis Less than Case 1, so only Check Biaxial Bending

$$\frac{f_{b1}}{F_{b1}'\left[1-\cancel{(f_c/F_{cE1})}\right]} + \frac{f_{b2}}{F_{b2}'\left[1-\cancel{(f_c/F_{cE2})}-(f_{b1}/F_{bE})^2\right]} \le 1.0$$

0 0

Adjustment Factors:

$C_D = 1.6$ (wind)
$C_V = 1.0$
$C_L = 0.95$

$F_{b1}' = 2400(1.6)(0.95) = 3648$ psi

$$f_{b1} = \frac{M}{S} = \frac{29812.05 \times 12}{155.7} = 2297.65 \text{ psi}$$

$F_{b2}' = 1450(1.6)(0.95) = 2204$ psi

$$f_{b2} = \frac{M}{S} = \frac{3149.28 \times 12}{59.10} = 639.45 \text{ psi}$$

$F_{bE} = 5649.77$ psi

$$\frac{2297.65}{3648} + \frac{639.45}{2204\left(1-\left(2297.65/5649.77\right)^2\right)} = 0.63 + 0.35 = 0.98 \le 1.0 \checkmark$$

→ $5\frac{1}{8}" \times 13\frac{1}{2}"$ GLB 24F-V4 works for Case 2

Case 3

$$M_{ux} = \frac{3978(18)}{4} + \frac{144.5(18)^2}{8} = 23{,}753.25 \text{ lbft}$$

$$M_{uy} = \frac{0.6(172.8 \text{ plf})(18)^2}{8} = 4199.04 \text{ lbft}$$

$$V_u = \frac{3978}{2} + \frac{144.5(18)}{2} = 3289.5 \text{ lbs}$$

- Shear and Moment about Strong Axis less than Case 1, so only check Biaxial Bending

Adjustment Factors:

$C_D = 1.6$ (wind)

$C_V = 1.0$

$C_L = 0.95$

$$F_{b1}' = 2400(1.6)(0.95) = 3648 \text{ psi}$$

$$f_{b1} = \frac{M}{S} = \frac{23753.25 \times 12}{155.7} = 1830.69 \text{ psi}$$

$$F_{b2}' = 1450(1.6)(0.95) = 2204 \text{ psi}$$

$$f_{b2} = \frac{M}{S} = \frac{4199.04 \times 12}{59.10} = 852.6 \text{ psi}$$

$$F_{bE} = 5649.77 \text{ psi}$$

$$\frac{1830.69 \text{ psi}}{3648 \text{ psi}} + \frac{852.6 \text{ psi}}{2204\left(1-\left(\frac{1830.69}{5649.77}\right)^2\right)} = 0.50 + 0.43 = 0.93$$

$0.93 \leq 1.0$ ✓

$\rightarrow$ $\boxed{\text{Use } 5\tfrac{1}{8}'' \times 13\tfrac{1}{2}'' \text{ GLB 24F-V4}}$

Check Transfer Girder at the Loft

→ $5\frac{1}{8}'' \times 13\frac{1}{2}''$

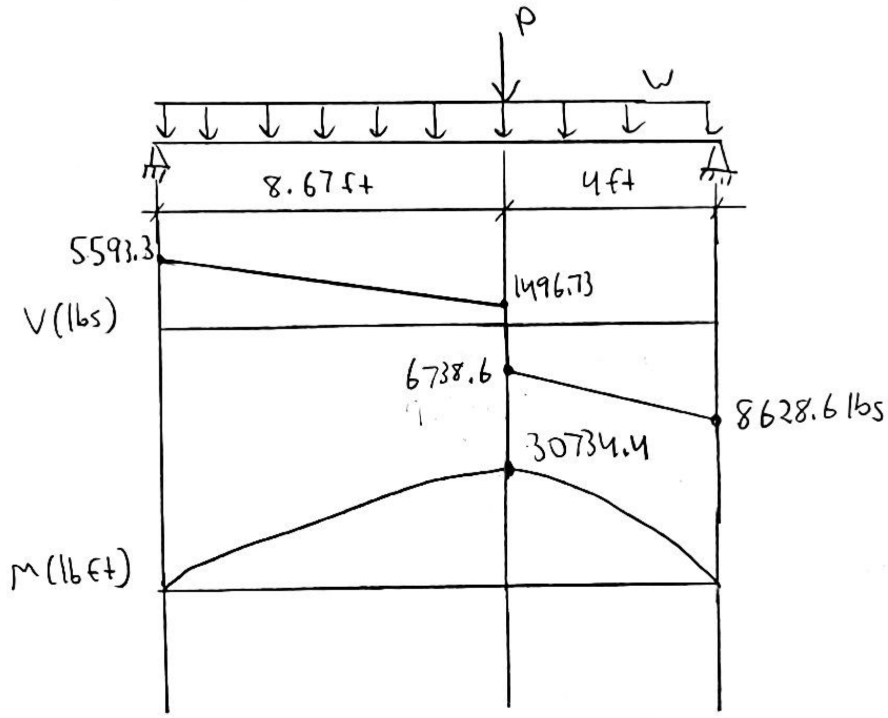
$P_{ridge} = 2462.1 + 5773.2 = 8235.3 \text{ lbs}$

Trib. Width $= \frac{15}{2} = 7.5 \text{ ft}$

$D_{Loft} = 23 \text{ psf}$

$L_L = 40 \text{ psf}$

$W = (23+40)(7.5) = 472.5 \text{ plf}$



Check Bending

$F_b = 2400 \text{ psi}$

Adjustment factors:

$C_D = 1.25$ $C_L (\text{every } 4 \text{ ft}) = 1.0$

$C_V = 1.0$

$F_b' = 2400(1.25) = 3000 \text{ psi}$

$f_b = \frac{M}{S} = \frac{30734.4 \times 12}{155.7} = 2368.74 \text{ psi}$ ✓

$DCR = \frac{2368.74}{3000} = \underline{0.789}$

Check Shear

$F_v' = 265psi(1.25) = 331.25psi$

$f_v = \frac{1.5V}{A} = \frac{1.5(8628.6)}{69.19} = 187.1\ psi$ ✓

$DCR = \frac{187.1}{331.25} = \underline{0.565}$

Check Deflection

$L/240 = \frac{12.67 \times 12}{240} = 0.63''$

Δ (from Table 3-23 of Steel Manual) =

$\frac{wx}{24EI}(L^3 - 2Lx^2 + x^3) + \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIL}$

$w_{50\%D} = (23/2 + 40)(7.5) = 386.25plf$

$\frac{(386.25 \times \frac{1}{12})(8.67 \times 12)}{24(1.8 \times 10^6)\ I}(152^3 - 2(152)(104)^2 + (104)^3) +$

$\frac{8235.3(48)(104)(104+2(48))\sqrt{3(104)(104+2(48))}}{27(1.8 \times 10^6)(152)\ I}$

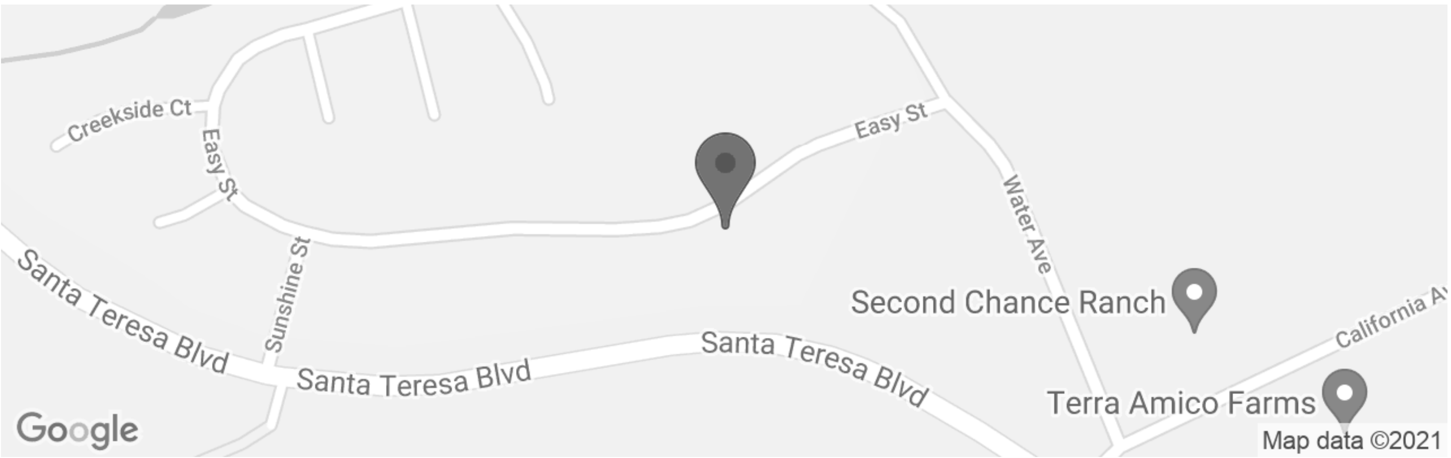
$= \frac{104.54}{I} + \frac{4.64}{I} = \frac{109.18}{1051} = 0.103'' < 0.63''$

$\rightarrow$ $\boxed{\text{use } 5\frac{1}{2}'' \times 13\frac{1}{2}'' \text{ GLB 24F-V4}}$

OSHPD

1125 Easy St, Morgan Hill, CA 95037, USA

Latitude, Longitude: 37.0895969, -121.6309392



Date	6/10/2021, 2:49:05 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.601	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.721	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	2.362	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.474	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.866	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.939	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.601	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.955	Mapped value of the risk coefficient at short periods

EXCEL HORIZONTAL FORCE DISTRIBUTION

Building Weight			
Loft	Area	PSF	Total
Floor weight	457.5	43	19672.5
Weight (kips)			**19.6725**
Roof	Area	PSF	Total
Roof Weight	1472.32	63	92756.16
Weight (kips)			**92.76**
Total Weight (kips)			112.43

North/South								
Level	Wx (kips)	hx (ft)	Wx*hx^k	Cvx = Wx*hx^k/Σ(wx*hx^k)	Fx = Cvx*V	Fx*ΣWx (kips)	Ax = Fx/Wx (%g)	Cs
Roof	112.43	14.25	1602.11	1.0000	0.185*W	20.80	0.1850	0.185
Σ	112.4		1602.11					

East/West								
Level	Wx (kips)	hx (ft)	Wx*hx^k	Cvx = Wx*hx^k/Σ(wx*hx^k)	Fx = Cvx*V	Fx*ΣWx (kips)	Ax = Fx/Wx (%g)	Cs
Roof	112.43	14.25	1602.11	1.0000	0.185*W	20.80	0.1850	0.185
Σ	112.4		1602.11					

1' Strip Weight E/W			
Garage and Loft (Grids B-C)	Area	PSF	Total
Loft weight	671	106	71126
Garage weight	409.2	63	25779.6
Total / 1' strip (kips/ft)			**6.46**
Great Room (Grid C-E)	Area	PSF	Total
Great Room Weight	511.9999	63	32256
Total / 1' strip (kips/ft)			**1.760**
wt x Ax			
	kips/ft	Ax	wt x Ax
Grids B-C	6.46	0.185	1.195
Grids C-E	1.76	0.185	0.326

1' Strip Weight N/S			
Garage	Area	PSF	Total
Floor weight	407	63	25641
Total / 1' strip (kips/ft)			**1.65**
Bedroom and Loft	Area	PSF	Total
Bedroom and Loft Weight	277.75	106	29442
Total / 1' strip (kips/ft)			**2.332**
Great Room and Loft	Area	PSF	Total
Loft	351.00	106	37206
Great Room	510	63	32130
Total / 1' strip (kips/ft)			**4.622**
wt x Ax			
	kips/ft	Ax	wt x Ax
Garage	1.65	0.185	0.306
Bedroom and Loft	2.33	0.185	0.431
Great Room and Loft	4.622	0.185	0.855

North/South Diaphragm Forces											Sds	I
Level	Wx (kips)	ΣWx	Fx	ΣFx	Fpx = ΣFx*wx / Σwx	Fpx.max = 0.4*Sds*I*Wx	Fpx.min = 0.2*Sds*I*Wx	Fpx.design (kips)	Ax	Scale Factor	1.2	1
Roof	112.43	112.43	20.80	20.80	20.80	53.97	26.98	26.9829	0.2400	1.30		
Σ	112.43											

North/South Diaphragm Forces											Sds	I
Level	Wx (kips)	ΣWx	Fx	ΣFx	Fpx = ΣFx*wx / Σwx	Fpx.max = 0.4*Sds*I*Wx	Fpx.min = 0.2*Sds*I*Wx	Fpx.design (kips)	Ax	Scale Factor	1.2	1
Roof	112.43	112.43	20.80	20.80	20.80	53.97	26.98	26.9829	0.2400	1.30		
Σ	112.43											

1' Strip Weight E/W			
Garage and Loft (Grids B-C)	Area	PSF	Total
Loft weight	671.00	106.00	71126.0
Garage weight	409.20	63.00	25779.6
Total / 1' strip (kips/ft)			**6.46**
Great Room (Grid C-E)	Area	PSF	Total
Great Room Weight	512.00	63.00	32256
Total / 1' strip (kips/ft)			**1.760**
wt x Ax			
	kips/ft	Ax	wt x Ax
Grids B-C	6.46	0.240	1.550
Grids C-E	1.76	0.240	0.422

1' Strip Weight N/S			
Garage	Area	PSF	Total
Floor weight	407.00	63.00	25641.0
Total / 1' strip (kips/ft)	-	-	**1.65**
Bedroom and Loft	Area	PSF	Total
Bedroom and Loft Weight	277.75	106.00	29442
Total / 1' strip (kips/ft)			**2.332**
Great Room and Loft	Area	PSF	Total
Loft	351.00	106.00	37206
Great Room	510.00	63.00	32130.0
Total / 1' strip (kips/ft)			**4.622**
wt x Ax			
	kips/ft	Ax	wt x Ax
Garage	1.65	0.240	0.397
Bedroom and Loft	2.33	0.240	0.560
Great Room and Loft	4.622	0.240	1.109

Earthquake Design

Base Shear

$V = C_s W$

From SEAOC MAPS,

Risk Category = II

Site Class = C

Address = 1125 Easy St. Morgan Hill

$S_{DS} = 1.2$

$S_{D1} = 0.56$

Table 12.2-1,

$R = 6.5$ (Both directions)

$I_e = 1.0$

$T = C_t h_n^x = 0.02(17ft)^{0.75} = 0.167s$

$C_s = \frac{S_{DS}}{(R/I_e)} = \frac{1.2}{(6.5/1.0)} = 0.185 < \frac{S_{D1}}{T(\frac{R}{I_e})} = \frac{0.56}{(0.167)(\frac{6.5}{1.0})} = 0.516$

$\rightarrow C_s = 0.185$

Seismic Weight

Roof Area = 1472.32 sqft.

D_L (including walls) = 63 psf

Roof Weight = 1472.32 sqft (63psf)/1000 = 92.76^k

Loft AREA = 15ft x (30.5ft) = 457.5 sqft.

D_L (Including Walls) = 43 psf

Loft Weight = 457.5 sqft (43psf)/1000 = 19.67^k

$W_{tot} = 92.76^k + 19.67^k = \underline{112.43^k}$

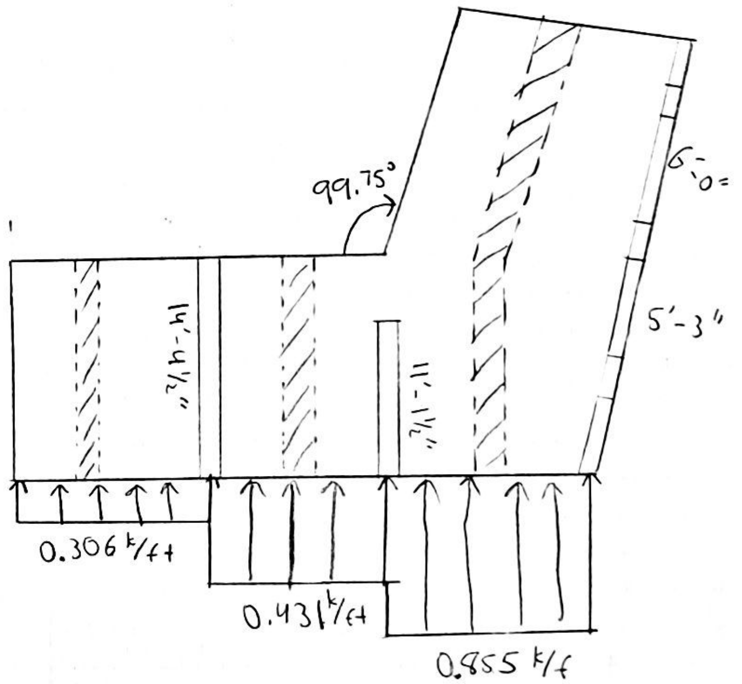
$V = C_s W = (0.185)(112.43^k)$

$\boxed{V = 20.8^k}$

* Assume One Story so apply V at the roof for vertical distribution

Sher Wall Force Distribution

N/S



Garage Shear Wall:

$$\Sigma M_{\text{Bedroom wall}} = (0.306\,k/ft)(15.5\,ft)\left(\frac{15.5}{2}+12.625\right) + 0.431\,k/ft(12.625)\left(\frac{12.625}{2}\right) - V_{Garage}(12.625) = 0$$

$$\boxed{V_{garage} = 10.38^k}$$

Bedroom Shear Wall:

From Left side →

$$\Sigma F = (0.306)(15.5) + (0.431)(12.625) - 10.38 - V_{bedroom} = 0$$

$$V_{bedroom} = -0.196^k$$

From right side →

$$V_{bedroom} = \frac{0.855(18)}{2} = 7.7^k$$

$V_{bedroom} = \boxed{7.7^k}$ — Conservative to ignore — reaction from Cantilever

Great Room Shear Wall:

5.48^k 9.75

$$V_{\text{Great room}} = \frac{\text{Adjacent Leg}}{\text{Cos}(\theta)} = \frac{7.7}{\text{Cos}(9.75)} = 7.81^k$$

$$\boxed{V_{\text{Great room}} = 7.81^k}$$

Shear Wall Nailing

N/S

GridLine 3: $V = 10.38^k$

$L = 14'\text{-}4\tfrac{1}{2}''$

$V_{ASD} = \frac{10.38}{14'\text{-}4\frac{1}{2}''}(0.7) = 0.505\ ^k/ft \rightarrow 505\ ^{lb}/ft$

Aspect Ratio $\rightarrow h = 14.5\ ft \rightarrow h/bs = \frac{14.5}{14'\text{-}4\frac{1}{2}''} = 1.0:1 \leq 2:1$ ✓

↳ midpoint of rake edge

→ $15/32''$ Struct. I, 8d Nails @ 3"

GridLine 5:

$V = 7.7^K$

$L = 11'\text{-}1\tfrac{1}{2}''$

$V_{ASD} = \frac{7.7}{11'\text{-}1\frac{1}{2}''}(0.7) = 0.485 \rightarrow 485\ ^{lb}/ft$

Aspect Ratio $\rightarrow h = 11.5\ ft = h/bs = \frac{11.5}{11.125} = 1.03:1 \leq 2:1$ ✓

→ $15/32''$ Struct. I, 8d Nails @ 3"

Grid Line 7

$V = 7.81^k$

$L_1 = 5'-3'' \longrightarrow h_1 = 14.5\ ft$ (midpoint of rake edge)

$L_2 = 6'-0'' \rightarrow h_2 = 11.5\ ft$

$V_{ASD} = \frac{7.81}{11.25}(0.7) = 0.486 \rightarrow 486\ ^{lb}/ft$

Aspect Ratio$_1 = \frac{14.5}{5.25} = 2.76:1 \nleq 2:1$

$\rightarrow VSP = 1.25 - 0.125(h/bs) = 1.25 - 0.125(2.76) = 0.905$

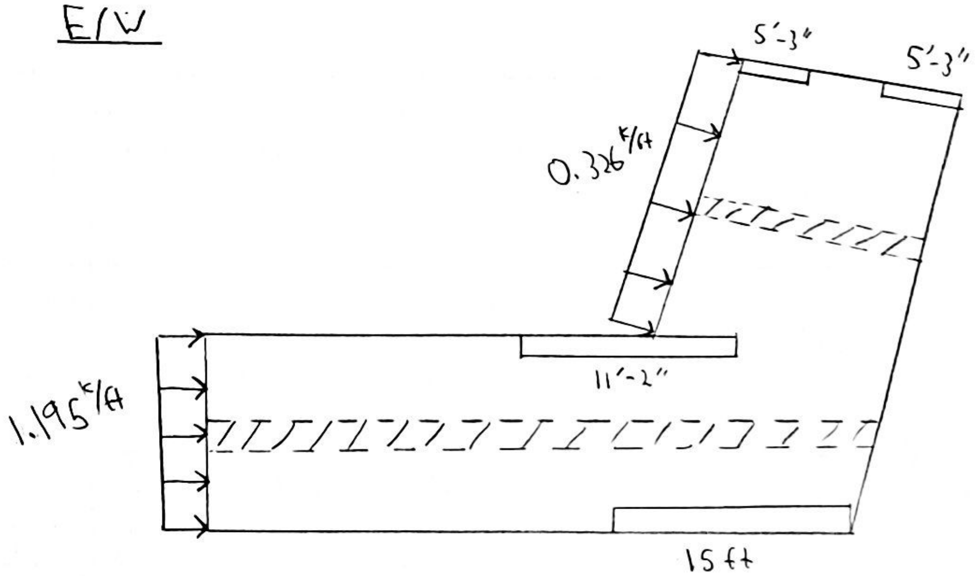
Aspect Ratio$_2 = \frac{11.5}{6.0} = 1.91:1 \leq 2:1$ ✓

$V_{ASD\ \text{Wall 1}} = \frac{486\ ^{lb}/ft}{0.905} = 537\ ^{lb}/ft$

→ Wall 1 → $15/32''$ Struct. I, 8d Nails @ 3"

Wall 2 → $15/32''$ Struct. I, 8d Nails @ 3"

E/W



Bottom Shear Wall:

$$V_{bottom} = \frac{1.195^{k}/ft\,(15\,ft)}{2} = \boxed{8.96^{k}}$$

Top Shear Wall:

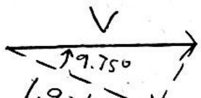
$$V_{top} : \frac{0.326^{k}/ft\,(18.33\,ft)}{2} = \boxed{2.99^{k}}$$

Middle Shear Wall:

From Bottom:

$$V_{middle} = \frac{1.195^{k}/ft\,(15ft)}{2} = 8.96^{k}$$

From Top:



$$V = \frac{2.99^{k}}{\cos(9.75)} = 3.03^{k}$$

$$V_{middle} = 8.96 + 3.03 = \boxed{12^{k}}$$

Shear Wall Nailing

E/W

Grid Line B

$V = 8.96^k$

$L = 15\,ft$

$v_{ASD} = \frac{8.96^k}{15\,ft}(0.7) = 0.42\,^k/ft = 420\,^{lb}/ft$

Aspect Ratio: $h = 11.5\,ft \rightarrow h/b_s = \frac{11.5}{15} = 0.77:1 \leq 2:1$ ✓

$\rightarrow$ $15/32''$ Struct. 1, 8d Nails @ 4''

Grid Line C

$V = 12^k$

$L = 11'-2''$

$v_{ASD} = \frac{12^k}{11'-2''}(0.7) = 0.752\,^k/ft = 752\,^{lb}/ft$

Aspect Ratio: $h = 11.5\,ft \rightarrow h/b_s = \frac{11.5}{11.167} = 1.03:1 \leq 2:1$ ✓

$\rightarrow$ $15/32''$ Struct. 1, 10d Nails @ 2''

Grid Line E

$V = 2.99^k$

$L = 5'-3'' \times 2 = 10'-6''$

$v_{ASD} = \frac{2.99^k}{10.5'}(0.7) = 0.20\,^k/ft = 200\,^{lb}/ft$

Aspect Ratio: $h = 14.5\,ft \rightarrow h/b_s = \frac{14.5}{5.25} = 2.76:1 \nleq 2:1$

$\rightarrow WSP = 1.25 - 0.125(h/b_s) = 1.25 - 0.125(2.76) = 0.905$

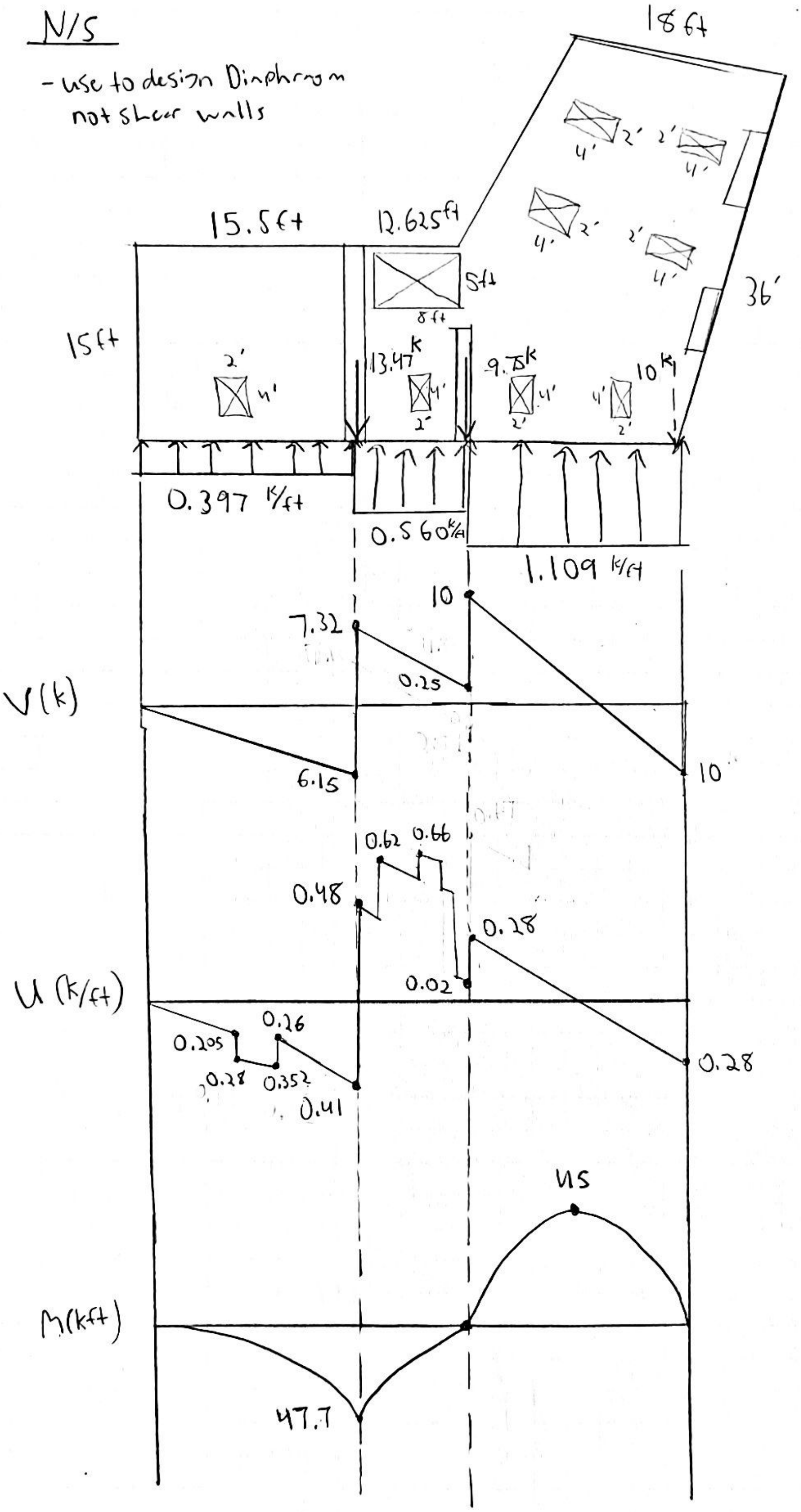
$v_{ASD} = \frac{200\,^{lb}/ft}{0.905} = 221\,^{lb}/ft$

$\rightarrow$ $15/32''$ Struct. 1, 8d Nails @ 4''

Diaphragm Forces

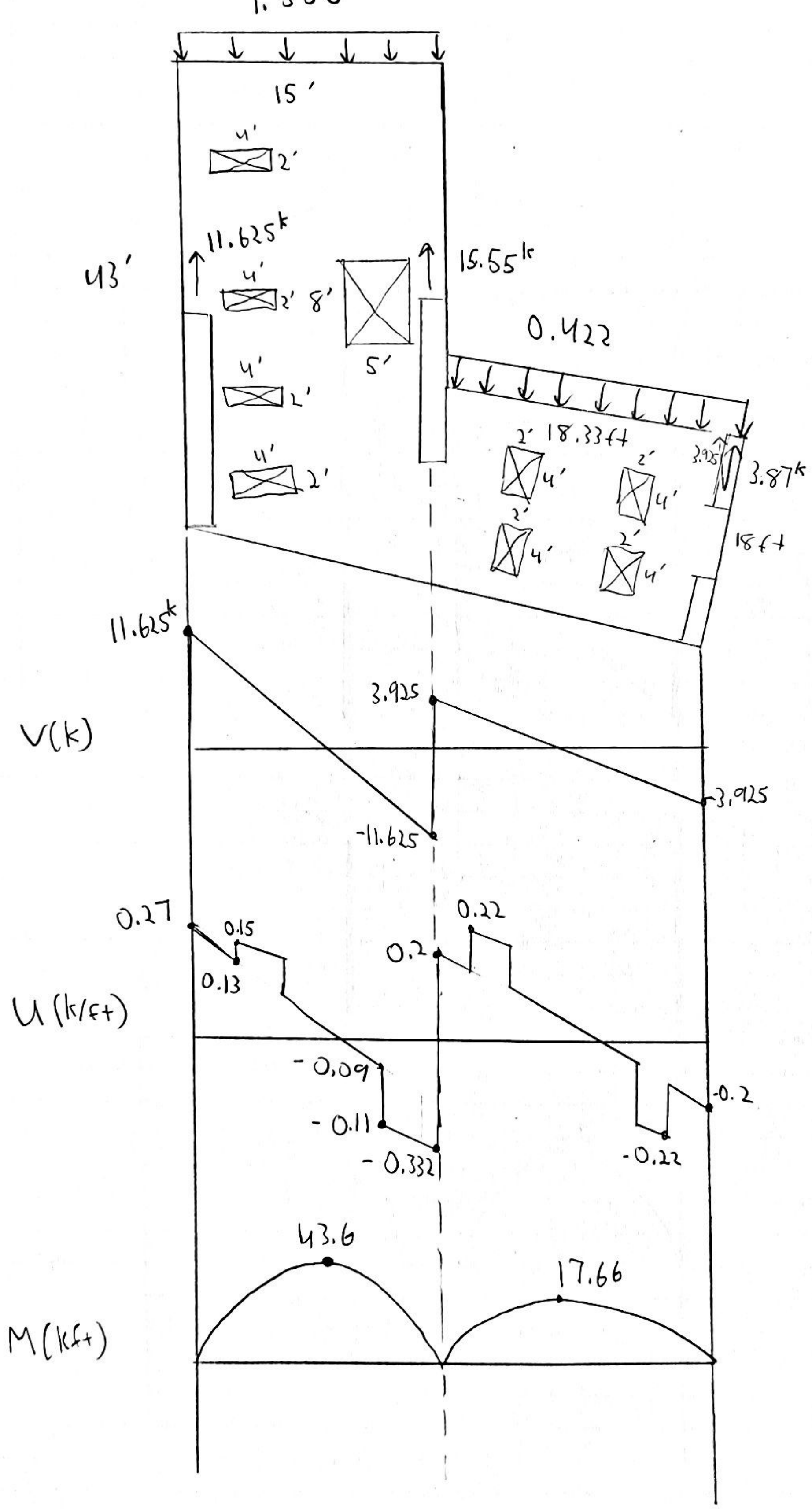
N/S

- use to design Diaphragm not shear walls



Diaphragm Forces

E/V



Diaphragm Design (Table 4.2A of NDS)

N/S

$u_{max} = 0.66$ k/ft

- Fully blocked from Decking
- Case 1

$u_{ASD} = 0.7(0.66^k/ft) = 0.462$ k/ft

→ 3/8", 8d Nails @ 2½" B.N. & C.N., 4" E.N.

$M = 540$ plf

* Only need Between Grids 3 - 5

Next Largest $u = 0.41$ k/ft

$u_{ASD} = 0.41(0.7) = 0.287$ k/ft

→ 3/8", 8d Nails @ 4" B.N. & C.N., 6" E.N.

$M = 360$ plf

E/W

$u_{max} = 0.332$ k/ft

- Fully Blocked
- Case 3

$u_{ASD} = 0.7(0.332) = 0.232$ k/ft

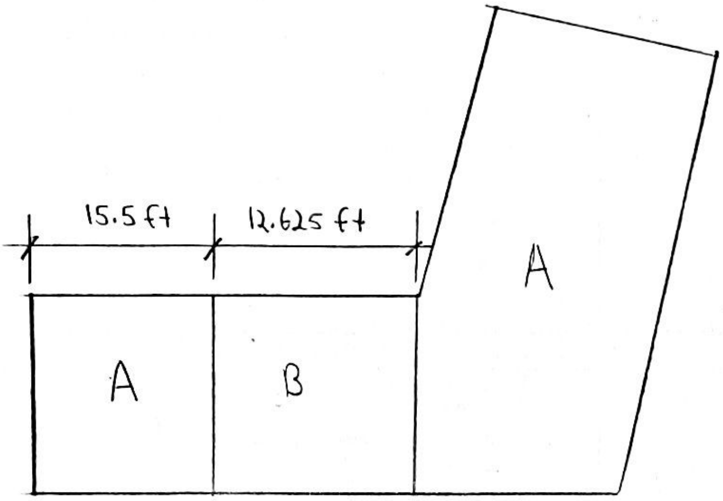
→ 3/8", 8d Nails @ 6" B.N. & C.N., 6" E.N.

$M = 270$ plf

* For simplicity use 4" B.N. & C.N., 6" E.N. to match N/S direction

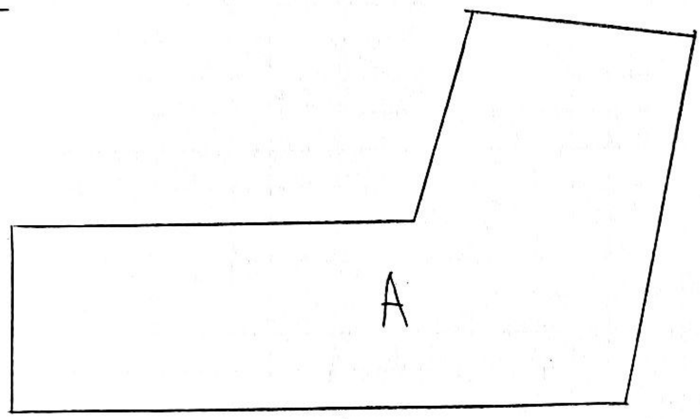
Diaphragm Nailing Key Plan

N/S



A = 3/8" Plywood, 8d Nails @ 4" B.N. & C.N., 6" E.N.
B = 3/8" Plywood, 8d Nails @ 2½" B.N. & C.N., 4" E.N.

E/W



A = 3/8" Plywood, 8d Nails @ 4" B.N. & C.N., 6" E.N.

Loft Diaphragm Design

1' strip Weight N/S

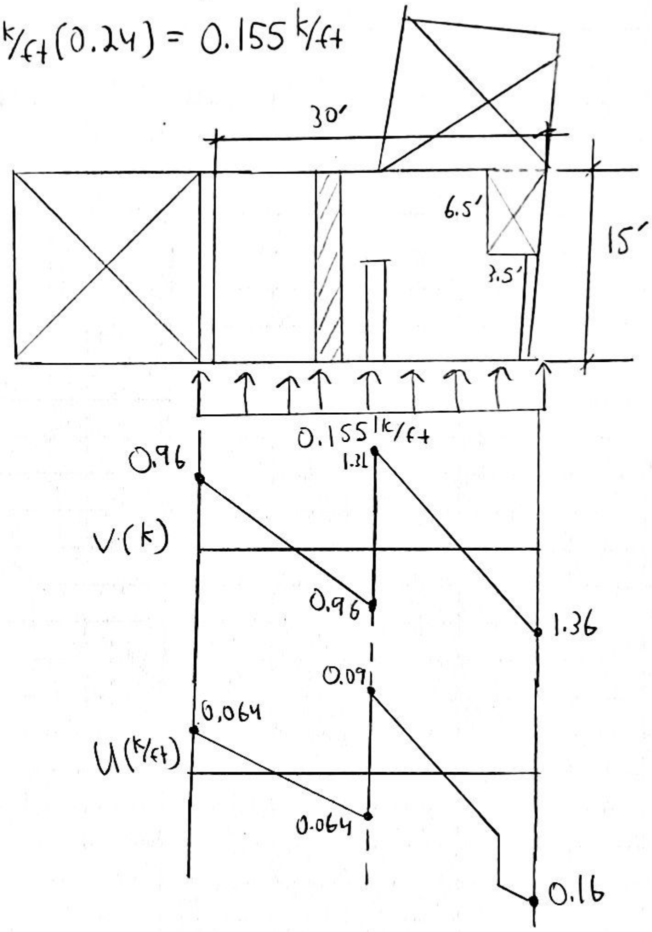
$A_x = 0.24$

$D_L = 43\,psf$

$Area = 15ft \times 30ft = 450ft^2$

$w = 43\,psf(450ft^2)/(1000 \cdot 30ft) = 0.645\ k/ft$

$w \cdot A_x = 0.645\ k/ft(0.24) = 0.155\ k/ft$



See Next Page →

1' Strip Weight E/W

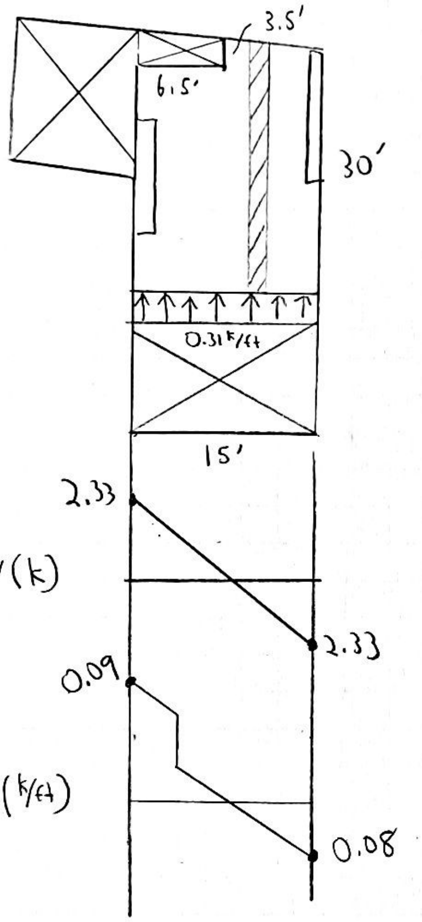
$A_x = 0.24$

$D_L = 43\ psf$

$Area = 15ft \times 30ft = 450\ ft^2$

$w = 43psf(450ft^2)/(1000 \cdot 15ft) = 1.29\ k/ft$

$w \cdot A_x = 1.29\ k/ft\ (0.24) = 0.31\ k/ft$



Loft Diaphragm

$U_{max} = 0.16\ k/ft$

-Fully Blocked

$U_{ASD} = 0.16\ k/ft\ (0.7) = 0.112\ k/ft = 112\ lbs/ft$

→ 3/8", 8d Nails @ 6" B.N. & C.N., 6" E.N.

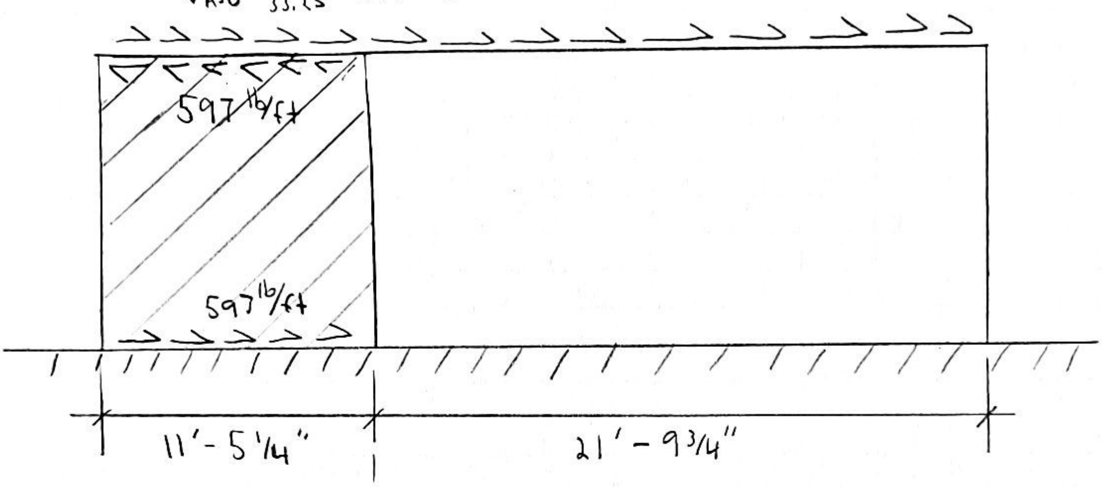
Collector Design

N/S

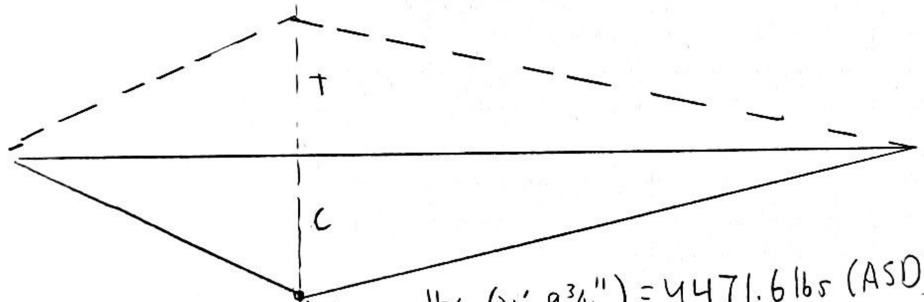
Gridline 3: → NO Collector as the Shear wall is the full Length of the diaphragm

Gridline 5:

$V_{ASD} = \frac{9.75}{33.25}\ ^{k}/_{ft}(0.7) = 0.205 \rightarrow 205\ ^{lb}/_{ft}$



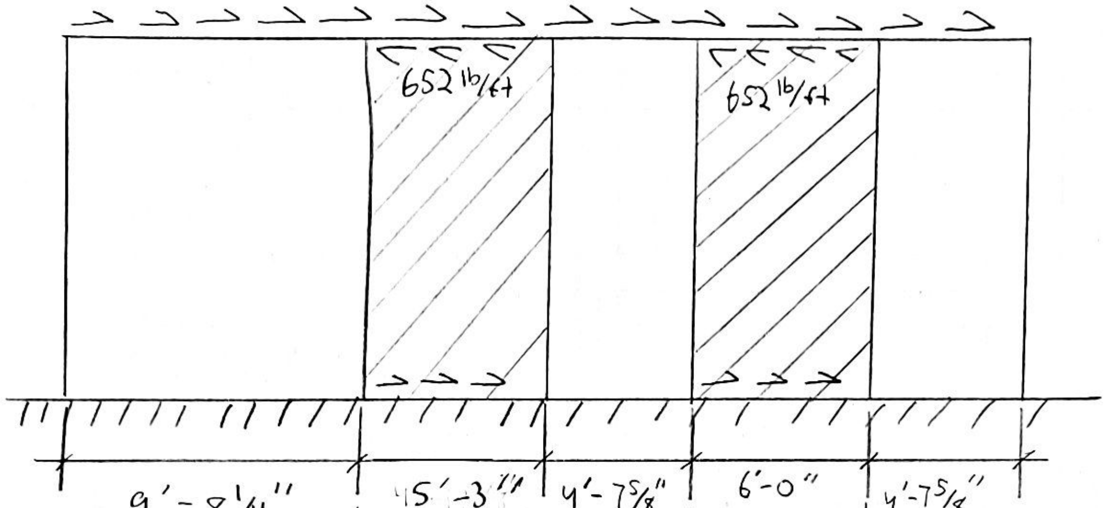
Axial Force Diagram



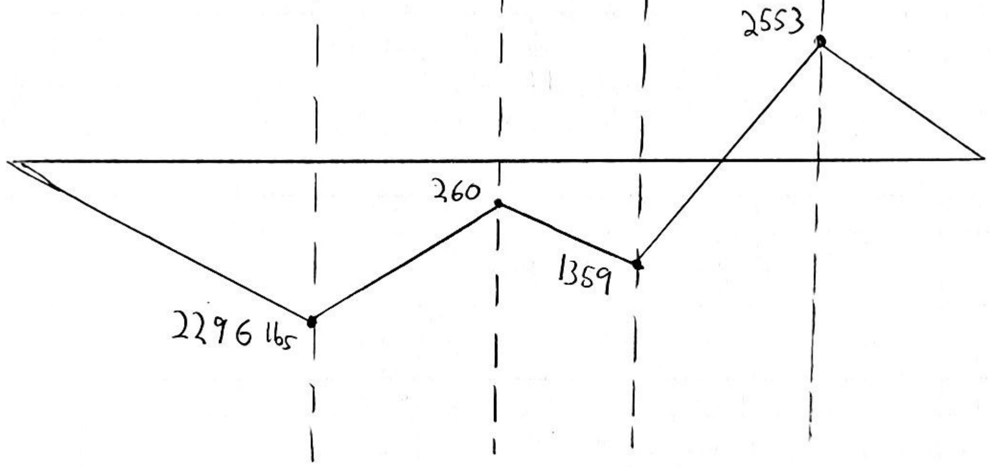
$250\ ^{lb}/_{ft}(21'\text{-}9\tfrac{3}{4}") = 4471.6\ lbs\ (ASD)$

Gridline 7

$V_{ASD} = \frac{10.15^k}{30\,ft}(0.7) = 0.237 \rightarrow 237\ ^{lb}/_{ft}$



Axial Force Diagram



Worst Case N/S Collector = Gridline 5

$P_{ASD} = 4{,}471.6\ lbs$

→ From Gravity Design, check $5\frac{1}{8}'' \times 13\frac{1}{2}''$ GLB

Load Combination

$1.0D + 0.7E_v + 0.7E_{mh}$ (Other Load Combos include L not Lr and these only take roof Loads)

$\rightarrow (1.0 + 0.14 S_{DS})D + \Omega_0 Q_{E\,ASD}$

$S_{DS} = 1.2$

Ω_0 (Table 12.2-1) $= 3 - 0.5 = 2.5$

↳ for Flexible diaphragm

Factored Loads

from Load Takeoff

$$(1.0+0.143(1.2))D = 1.172\,D = 1.172(39\,psf \times (7.5ft)) = 342.8\,lb/ft$$

$$M_u = \frac{342.8\,lb/ft\,(21.8125ft)^2}{8} = 20387.4\,lbft$$

$$\Omega_0 Q_{EASD} = 2.5(4{,}471.6\,lbs) = 11{,}179\,lbs$$

Check Combined Loading (Compression)

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{F'_{b1}\left(1-\frac{f_c}{F_{cE}}\right)} \le 1.0$$

Adjustment Factors:

$C_D = 1.6$

C_L (braced every 4 ft) $= 1.0$

$C_V = 0.98$

$C_P \rightarrow \; l_e/d = \frac{4ft \times 12}{5.125} = 9.37$ or $\frac{21.8125ft \times 12}{13.5} = 19.4$ ← Governs

$l_e/d = 19.4 < 50$ ✓

$$F_{cE} = \frac{0.822E_{min}'}{(l_e/d)^2} = \frac{0.822(0.95\times10^6)}{(19.4)^2} = 2074.88$$

$$F_c^* = 1650\,psi\,(1.6) = 2640\,psi$$

$$\frac{F_{cE}}{F_c^*} = \frac{2074.88}{2640} = 0.786$$

$$C_P = \frac{1+0.786}{2(0.9)} - \sqrt{\left[\frac{1+0.786}{2(0.9)}\right]^2 - \frac{0.786}{0.9}} = 0.992 - 0.333$$

$C_P = 0.66$

$$F'_c = 1650(1.6)(0.66) = 1742.4\,psi$$

$$f_c = \frac{P}{A} = \frac{11{,}179 \text{ lbs}}{69.19} = 161.57 \text{ psi}$$

$$F_b' = 2400(0.98)(1.6) = 3763.2 \text{ psi}$$

$$f_b = \frac{M}{S} = \frac{20387.4 \text{ lbft} \times 12}{155.7} = 1571.3 \text{ psi}$$

$$\rightarrow \left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left(1-\frac{f_c}{F_{cE}}\right)} = \left(\frac{161.57}{1742.4}\right)^2 + \frac{1571.3}{(3763.2)\left(1-\frac{161.57}{2074.88}\right)} =$$

$$= 0.0086 + 0.453 = 0.46 \leq 1.0 \checkmark$$

Check Combined Loading (Tension)

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0 \quad \text{and} \quad \frac{f_b - f_t}{F_b^{**}} \leq 1.0$$

$\hookrightarrow F_b$ w/o C_L $\qquad$ $\llcorner F_b$ w/o C_V

$F_t = 1100$ psi

$F_b = 2400$ psi

$C_D = 1.6$

$C_V = 0.98$

$C_L = 1.0$

$F_t' = 1100 \text{ psi}(1.6) = 1760$ psi

$F_b^* = 2400 \text{ psi}(1.6)(0.98) = 3763$ psi

$F_b^{**} = 2400 \text{ psi}(1.6) = 3840$ psi

$$f_t = \frac{P}{A} = \frac{11{,}179 \text{ lbs}}{69.19} = 161.57 \text{ psi}$$

$$f_b = \frac{M}{S} = \frac{20387.4 \text{ lbft} \times 12}{155.7} = 1571.3$$

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} = \frac{161.57}{1760} + \frac{1571.3}{3763} = 0.51 \leq 1.0 \checkmark$$

$$\frac{f_b - f_t}{F_b^{**}} = \frac{1571.3 - 161.57}{3840} = 0.37 \leq 1.0 \checkmark$$

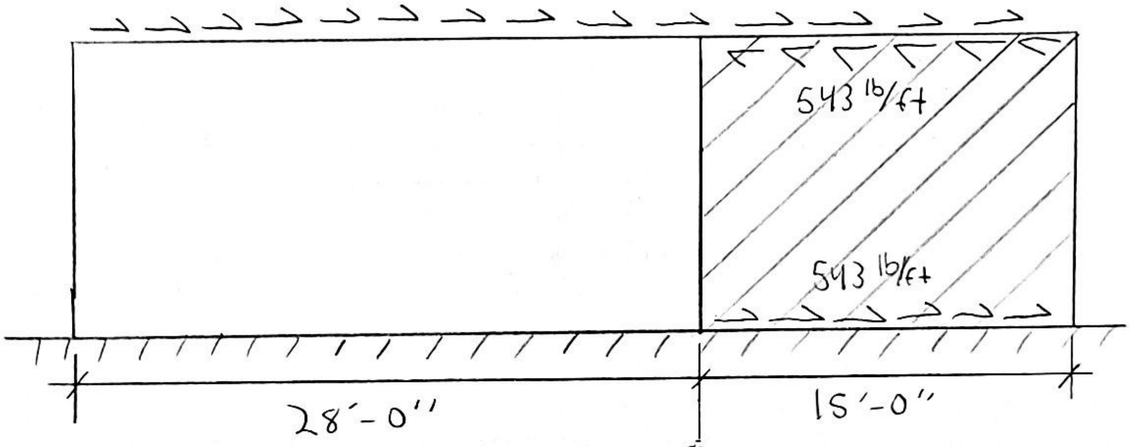
$\boxed{\text{Can use } 5\tfrac{1}{8}'' \times 13\tfrac{1}{2}'' \text{ for Collectors in N/S}}$

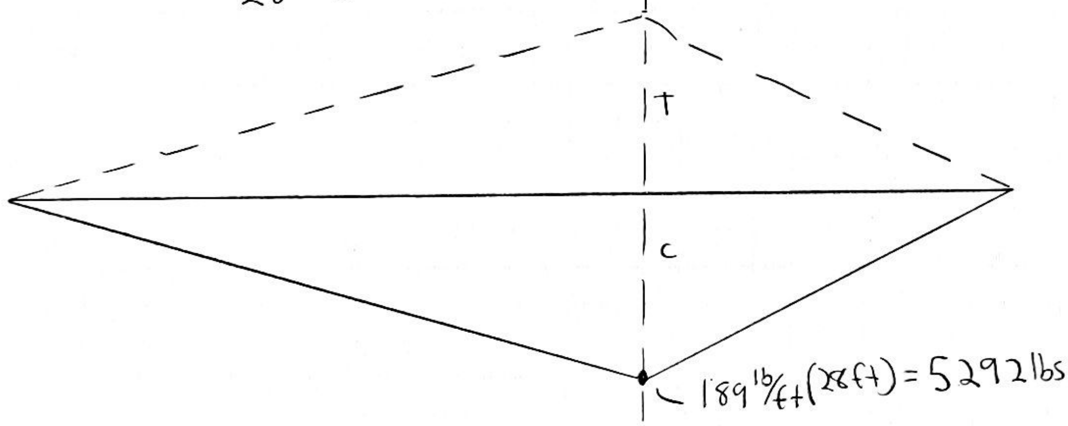
Collector Design

E/W

Gridline B

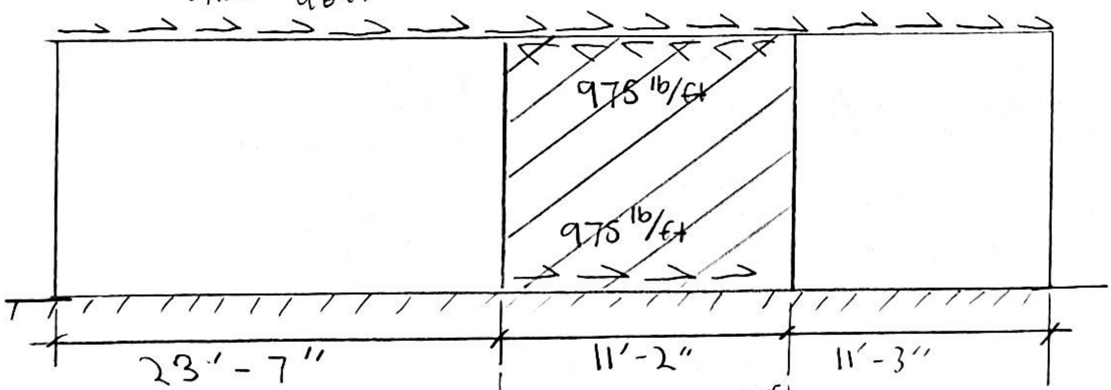
$$V_{ASD} = \frac{11.625}{43}(0.7) = 0.189\ ^{k}/_{ft} = 189\ ^{lb}/_{ft}$$

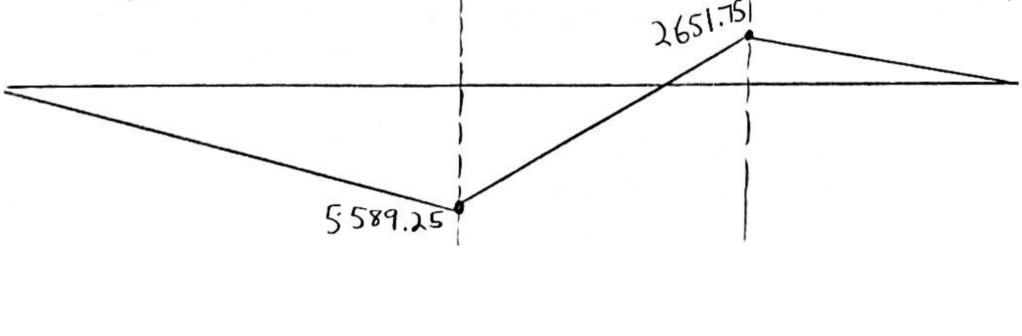




GridLine C

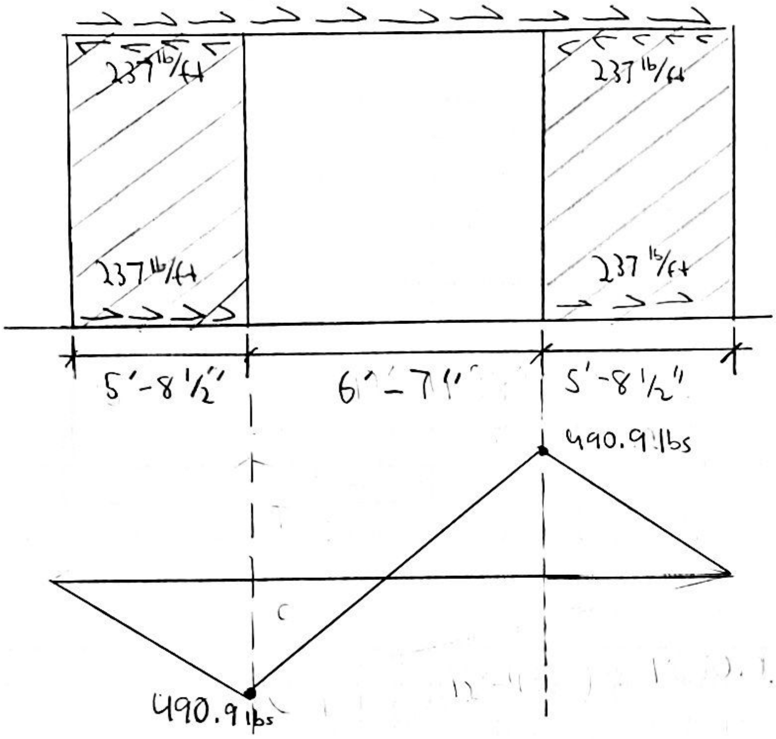
$$V_{ASD} = \frac{15.55^{k}}{46\,ft}(0.7) = 0.237 = 237\ ^{lb}/_{ft}$$





Gridline E

$V_{ASD} = \frac{3.87}{18}(0.7) = 0.151 \rightarrow 151\ ^{lb}/_{ft}$



Worst Case E/W Collector = Gridline C

$P_{ASD} = 5589.25 \text{ lbs}$

→ Check $5\frac{1}{8}" \times 13\frac{1}{2}"$ GLB

From N/S Direction;

$1.172D = 1.172(39psf(7.75\ ft)) = 354.24\ ^{lb}/_{ft}$

$M_u = \frac{354.24\ ^{lb}/_{ft}\ (23.583ft)^2}{8} = 24{,}627.4\ lbft$

$\Omega_o Q_{EASD} = 2.5(5589.25 lbs) = 13973.1\ lbs$

Check Combined Loading (compression)

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{F'_{b1}\left(1 - \frac{f_c}{F_{cE}}\right)} \leq 1.0$$

Adjustment Factors:

$C_D = 1.6$

C_L (braced every 4 ft) $= 1.0$

$C_V = 0.98$

$C_P \rightarrow l_e/d = \dfrac{23.583 \times 12}{13.5} = 21 \leq 50$ ✓

$$F_{cE} = \frac{0.822 E_{min}}{(l_e/d)^2} = \frac{0.822(0.95 \times 10^6)}{(21)^2} = 1770.75$$

$F_c^* = 1650\text{psi}(1.6) = 2640\text{ psi}$

$$\frac{F_{cE}}{F_c^*} = \frac{1770.75}{2640} = 0.67$$

$$C_P = \frac{1+0.67}{2(0.9)} - \sqrt{\left[\frac{1+0.67}{2(0.9)}\right]^2 - \frac{0.67}{0.9}} = 0.93 - 0.34 = 0.59$$

$F_c' = 1650(1.6)(0.59) = 1557.6\text{ psi}$

$$f_c = \frac{P}{A} = \frac{13973.1\text{ lbs}}{69.19} = 201.95\text{ psi}$$

$F_b' = 2400(0.98)(1.6) = 3763.2\text{ psi}$

$$f_b = \frac{M}{S} = \frac{24,627.4\text{ lbft} \times 12}{155.4} = 1898\text{ psi}$$

$$\rightarrow \left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left(1 - \frac{f_c}{F_{cE}}\right)} = \left(\frac{201.95}{1557.6}\right)^2 + \frac{1898}{3763.2\left(1 - \frac{201.95}{1770.75}\right)}$$

$$= 0.017 + 0.57 = 0.586 \leq 1.0 \checkmark$$

<u>Check Combined Loading (Tension)</u>

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0 \quad \text{and} \quad \frac{f_b - f_t}{F_b^{**}} \leq 1.0$$

$F_t = 1100\text{ psi}$

$F_b = 2400\text{ psi}$

Adjustment factors

$C_D = 1.6$

$C_V = 0.98$

C_L (braced every 4 ft) $= 1.0$

$F'_t = 1100\text{ psi}(1.6) = 1760\text{ psi}$

$F_b^* = 2400\text{ psi}(1.6)(0.98) = 3763\text{ psi}$

$F_b^{**} = 2400\text{psi}(1.6) = 3840\text{ psi}$

$f_t = \frac{P}{A} = \frac{13973.1\text{lbs}}{69.19} = 201.95\text{psi}$

$f_b = \frac{M}{S} = \frac{24627.4\text{ lbft} \times 12}{155.7} = 1898\text{ psi}$

$\frac{f_t}{F'_t} + \frac{f_b}{F_b^*} = \frac{201.95}{1760} + \frac{1898}{3763} = 0.62 \leq 1.0$ ✓

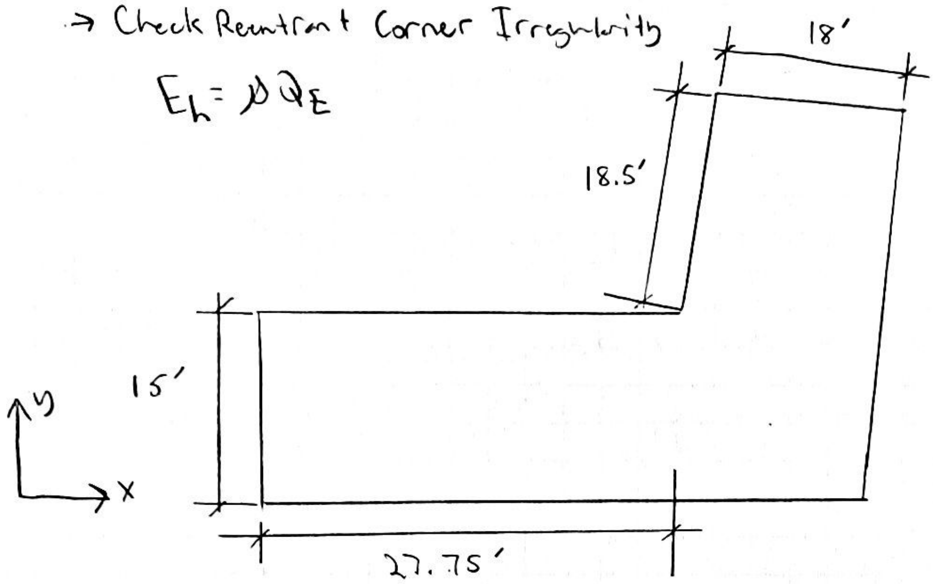
$\frac{f_b - f_t}{F_b^{**}} = \frac{1898 - 201.95}{3840} = 0.44 \leq 1.0$ ✓

Can use 5 1/8" x 13 1/2" for Collectors in E/W

Shear Wall Connectors

→ Check Reentrant Corner Irregularity

$E_h = \rho Q_E$



In x-direction,

$$\frac{18\,ft}{(27.75+18)ft} = 0.39 \rightarrow 39\%$$ of plan dimension is beyond the reentrant corner

$\rightarrow \underline{\rho = 1.3}$

Shear Clip Spacing

– Use Simpson A35 clips

From Catalog → $V_{allow} = 650\ lbs$ per clip

Worst Case Boundary Shear $U_{max} = 0.48\ k/ft \rightarrow 480\ lbs/ft$

$$U_{ASD} = 0.7\,(480\ lb/ft)(\rho) = 0.7\,(480\ lbs/ft)(1.3) = 436.8\ lb/ft$$

$$\frac{V_{allow}}{S} \cdot 12\ in/ft = U_{ASD} = \frac{650\ lbs}{S} \cdot 12\ in/ft = 436.8\ lbs/ft$$

$\rightarrow S = 17.86'' \rightarrow$ Round to $\underline{S = 16''}$

– Another Option is Strong-Drive SDWS TIMBER Screws

From Simpson Catalog →

blocking = 2x6

Thread Length = 2.75"

→ Min. Length for full embeddment = 8.75"

→ Use 0.220 x 9"

$V_{allow} = 395\ lbs\,(1.6) = 632\ lbs$

$u_{ASD} = 436.8\ ^{lb}/_{ft}$

$$\frac{V_{allow}}{s} \cdot 12\ ^{in}/_{ft} = u_{ASD} = \frac{632\ lbs}{s} \cdot 12\ ^{in}/_{ft} = 436.8\ ^{lbs}/_{ft}$$

$\rightarrow s = 17.36'' \rightarrow$ Round to $\underline{s = 16''}$

May use A35 clips and cover with blocking or use Strong-Drive Timber Screws w/ $s = 16''$

Anchor Bolt Spacing

Table 12E in NDS,

- Douglas fir Larch
- $t = 1.5''$
- use 5/8" Bolts

$Z_{11} = 930\ lbs$ (6" Anchor Embed Min.)

Adjustment Factors:

$C_D = 1.6$ $\quad C_{tn} = 1.0$

$C_g = 1.0$ $\quad C_{di} = 1.0$

- Estimate spacing in order to check C_Δ and C_{eg}

$u_{ASD} = 975\ ^{lbs}/_{ft}$ (worst case)

$$s = \frac{Z_{11}^*}{u_{ASD}} \times 12\ ^{in}/_{ft} = \frac{930\ lbs\,(1.6)}{975\ ^{lbs}/_{ft}} \cdot 12\ ^{in}/_{ft} = 18.3'' \rightarrow s \approx 18''$$

$\underline{C_\Delta}$

Edge Dist. $\approx 5.5''$ (6x6 Posts)

Table 12.5.1A $\rightarrow$ Min. Edge Dist. $= 7D = 7(5/8'') = 4.375'' < 5.5''$ ✓

Table 12.5.1B $\rightarrow$ Min. Spacing $= 4D = 4(5/8'') = 2.5'' < 18''$ ✓

$\rightarrow C_\Delta = 1.0$

$\underline{C_{eg}}$

Table 12.5.1C $\rightarrow$ Min. Edge Dist. $= 1.5D = 1.5(5/8'') = 0.94'' < 5.5''$ ✓

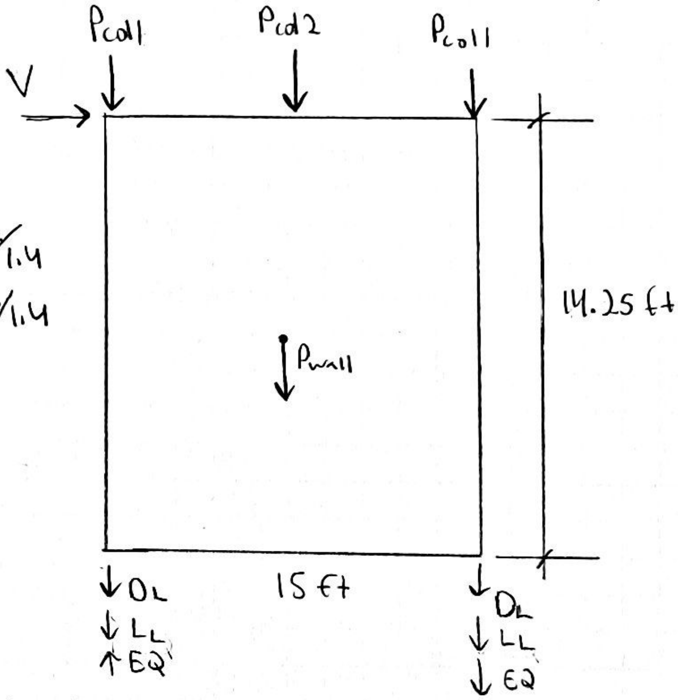
Table 12.5.1D $\rightarrow$ Min. Spacing $= 5D = 5(5/8'') = 3.125'' < 18''$ ✓

$\rightarrow C_{eg} = 1.0$

$\boxed{\text{Use 5/8" Anchor Bolts @ 18" O.C.}}$

Shear Wall Overturning

GridLine 3



Load Combinations

$T = (0.9 - 0.143 S_{DS}) D + \rho Q_E / 1.4$

$C = (1.0 + 0.143 S_{DS}) D + L + \rho Q_E / 1.4$

$D_L = (45 + 28) = 73\ psf$

$D_{wall} = 17\ psf$

$L = 40\ psf$

$V = 10.38^k$

Trib. Area for Col. 1 $= \left(\frac{7.5}{2} + 3\right)\left(\frac{15.5}{2} + \frac{12.625}{2}\right) = 94.92$ sqft.

Trib Area for Col. 2 $= (7.5ft)\left(\frac{15.5}{2} + \frac{30\ ft}{2}\right) = 170.63$ sqft.

Trib Area for $L_L = \left(\frac{12.625}{2}\right)\left(\frac{15}{2}\right) = 47.34$ sqft.

Tension

$T = (0.9 - 0.143 S_{DS}) D + \rho Q_E / 1.4$

$D = 73\ psf (94.92\ sqft) + \frac{73\ psf\ (170.63)}{2} = 13{,}157.2\ lbs$

$Q_E \rightarrow T/C$ Couple $= \frac{VH}{W} = \frac{10.38^k (14.25ft)}{15\ ft} = 9861\ lbs$

$T = (0.9 - 0.143(1.2))(13{,}157.2\ lbs) - (0.7)(1.3)(9861)$

$T = 610.2\ lbs$

$\rightarrow$ No tension Capacity needed

Compression

$$C = (1 + 0.143\, S_{DS})D + L + \rho Q_E / 1.4$$

$D = 13{,}157.2$ lbs

$L = 40psf(47.34\ sq.ft.) = 1893.6$ lbs

$Q_E = 9861$ lbs

$C = (1 + 0.143(1.2))(13{,}157.2\ lbs) + 1893.6\ lbs + (1.3)(0.7)(9861\ lbs)$

$C = 26{,}282$ lbs

→ Sawn Lumber DF-L NO. 1, 6x6

$F_c = 925$ psi

$C_D = 1.6$

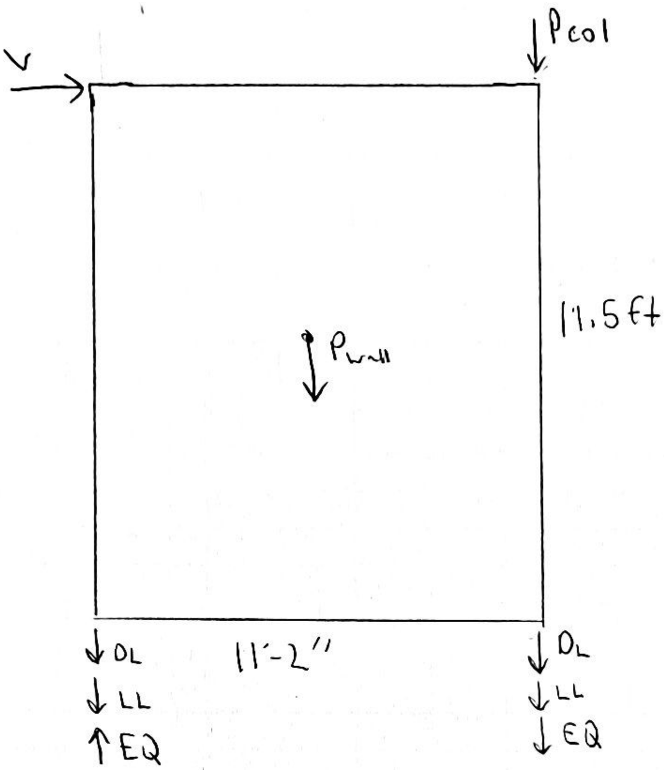
$C_P = 0.695$

$F'_c = 925(1.6)(0.695) = 1028.6\ psi$

$f_c = \frac{P}{A} = \frac{26{,}282\ lbs}{30.25\ in^2} = 868.83\ psi < F'_c = 1028.6\ psi$ ✓

$\boxed{\text{Use } 6\times6 \text{ DF-L NO. 1 Chord}}$

GridLine 5



$D_L = 73$ psf
$D_{wall} = 17$ psf
$L = 40$ psf
$V = 7.7^k$

Trib Area of Columns = $\left(\frac{18}{2} + \frac{12.625}{2}\right)\left(\frac{15}{2}\right) = 114.84$ sq.ft

T/C couple = $7.7^k(11.5ft)/11.167ft = 7.93^k$

Tension

$T = (0.9 - 0.143 S_{DS})D + \rho Q_E/1.4$

D => Assuming Only One Side Carries Column dead Load, so worst case tension does not use col. weight, only self weight

$= 17psf(11.5ft)(11.167ft)/2 = 1092$ lbs

$T = (0.9 - 0.143(1.2))(1092) - (0.7)(1.3)(7.93^k)$

$T = -6420.88$ lbs

From Simpson Catalog,

Try HD7B w/ (3) 3/4" Bolts and a 6x6 Chord

Compression

$$C = (1 + 0.14 S_{DS})D + L + \rho Q_E/1.4$$

$D = 73\text{psf}(114.84 \text{ sq.ft}) + 2183.15 \text{ lbs} = 10566.5 \text{ lbs}$

$L = 40\text{psf}(114.84 \text{ sq.ft}) = 4593.6 \text{ lbs}$

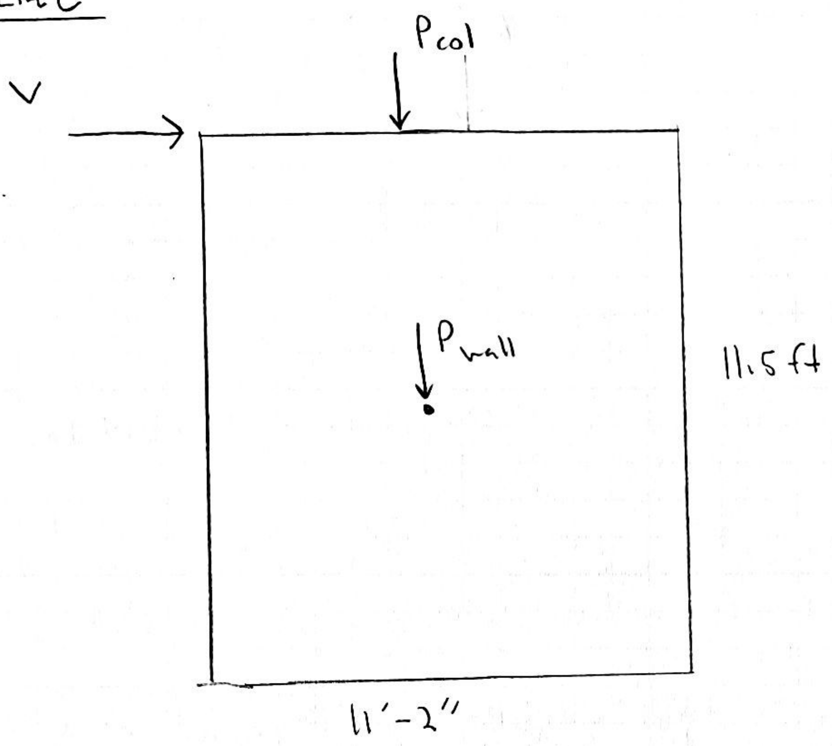
$Q_E = 7.93^k$

$C = (1.0 + 0.143(1.2))(10566.5) + 4593.6 + (1.3)(0.7)(7.93^k)$

$C = 24{,}189.6 \text{ lbs}$ < The compression force for chord on Gridline 3, so by inference 6x6 works

$\boxed{\text{use 6x6 DF-L No.1 chords w/ HD7B w/ (3) 3/4" Bolts}}$

Grid Line C



$V = 12^k$

$D_L = 73\text{ psf}$

$D_{wall} = 17\text{ psf}$

$L = 40\text{ psf}$

$$\text{Trib. Area for Col.} = \left(\frac{15}{2} + \frac{18.5}{2}\right)\left(\frac{12.625}{2} + \frac{18}{2}\right) = 256.48\text{ sq.ft.}$$

$$T/c\ \text{couple} = \frac{12000\text{ lbs}(11.5\text{ ft})}{11.167\text{ ft}} = 12{,}357\text{ lbs}$$

Tension

$$T = (0.9 - 0.143\, S_{DS})D + \rho Q_E/1.4$$

$$D = (256.48)(73)/_2 + 17\text{ psf}(11.5\text{ ft})(11.167\text{ ft})/_2 = 10{,}453.1\text{ lbs.}$$

$$T = (0.9 - 0.143(1.2))(10{,}453.1\text{ lbs}) - (1.3)(0.7)(12{,}357\text{ lbs})$$

$$T = -3630.83\text{ lbs}$$

→ Try HDSB w/ (2) 3/4" Bolts and a 6x6 Chord

Compression

$$C = (1 + 0.14 S_{DS})D + L + \rho Q_E/1.4$$

$D = 10,453.1$ lbs

$L = 40\text{psf}\left(\frac{15}{2}\right)\left(\frac{13.625}{2} + \frac{18}{2}\right) = 4593.75$ lbs

$Q_E = 12,357$ lbs

$$C = (1 + 0.143(1.2))(10,453.1) + (4593.75) + 12,357$$

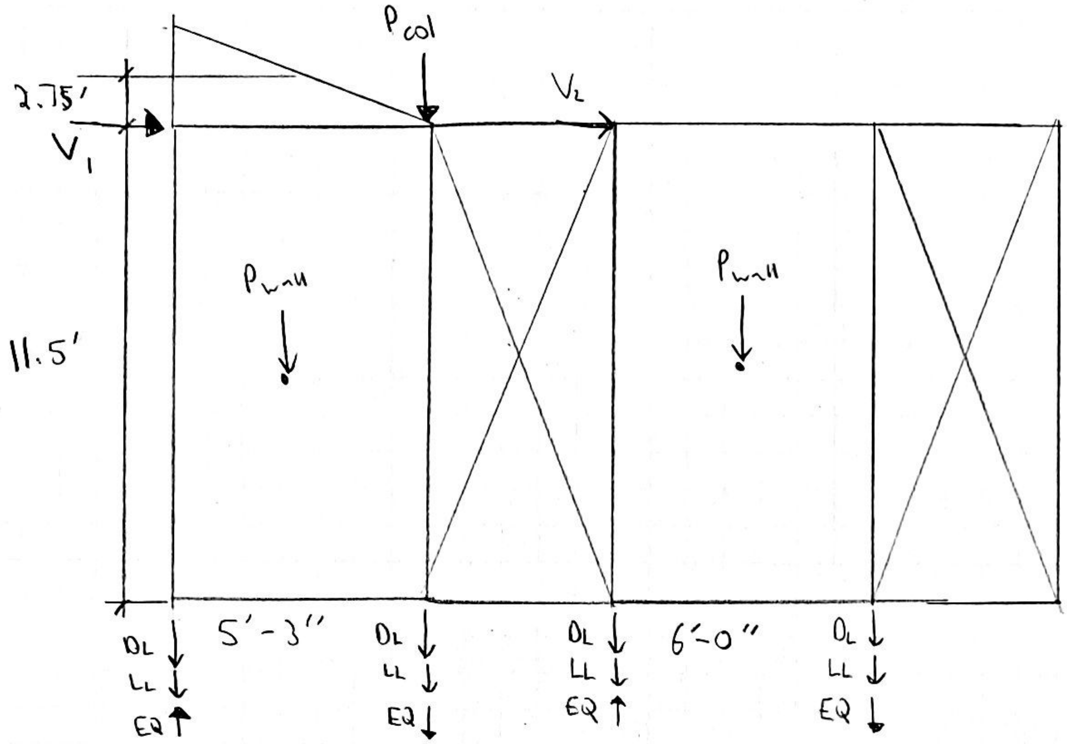
$C = 29,197.6$ lbs

$\rightarrow F_c' = 1028.6$ psi (from Gridline 5 chord design)

$$f_c = \frac{P}{A} = \frac{29,197.6}{30.25} = 965.2 \text{ psi} \leq F_c' = 1028.6 \text{ psi} \checkmark$$

$\boxed{\text{Use } 6\times6 \text{ DF-L No. 1 Chord w/ HD5B w/ (2) } 3/4'' \text{ Bolts}}$

Grid Line 7



$D_L = 73\,psf$

$D_{wall} = 17\,psf$

$L = 40\,psf$

$V_1 = 652\,lb/ft\,(5.25\,ft) = 3423\,lbs$ (ASD)

$V_2 = 652\,lb/ft\,(6\,ft) = 3912\,lbs$ (ASD)

Trib Area for Col. $= \left(\frac{18.5}{2} + \frac{15}{2}\right)\left(\frac{18}{2}\right) = 150.75\,ft^2$

Trib Area for L_L $= \left(\frac{15}{2}\right)(18/2) = 67.5\,ft^2$

Tension

$T = (0.9 - 0.143\,S_{DS})D + \Omega Q_E/1.4$

Worst case Dead Load is only from the wall

$D = \underline{17\,psf\,(5.25 \times 14.25)} = 635.9\,lbs$
2

Worst Case is Shorter Wall $\rightarrow$ $Q_E = T/C$ couple $= \frac{VH}{W} = \frac{3423\,lbs\,(14.25\,ft)}{5.25\,ft} = 9291\,lbs$ (ASD)

$T = (0.9 - 0.143(1.2))(635.9\,lbs) - 1.3\,(9291\,lbs)$

$T = 11615.11\,lbs$

From Simpson Catalog,

Try HD12 w/ (4) 1" Bolts and a 6x6 Chord

Compression

$$C = (1+0.14S_{DS})D + L + \rho Q_E/1.4$$

$D = 73\,psf\,(150.75\,ft^2) + 635.9\,lbs = 11{,}640.65\,lbs$

$L = 40\,psf\,(67.5\,ft^2) = 2700\,lbs$

$Q_E = 9291\,lbs$ (ASD)

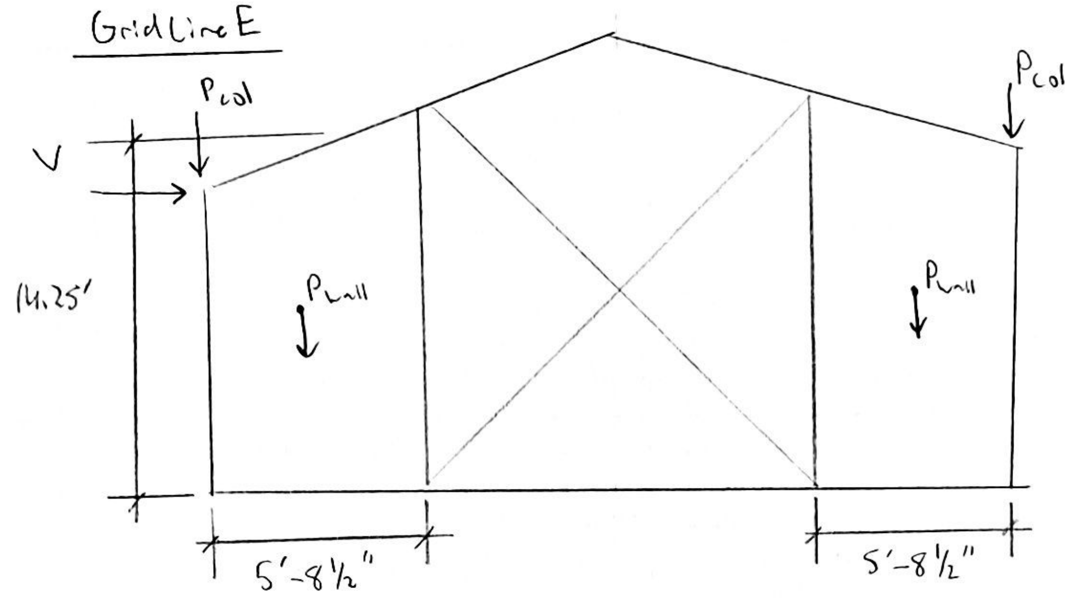
$$C = (1+0.14(1.2))(11{,}640.65\,lbs) + 2700 + 1.3(9291\,lbs)$$

$C = 28{,}416.48\,lbs$

→ Less compression than the chords on Grid Line C so 6x6 are adequate by inspection

$\boxed{\text{use 6x6 DF-L NO.1 chord w/ HD12 w/ (4) 1" Bolts}}$

Grid Line E



$V = 237\,lb/ft\,(5.708') = 1352.9\,lbs$ (ASD)

$D_L = 73\,psf$

- No Live Load

$D_{wall} = 17\,psf$

Trib. Area for Col. $= \left(\frac{18.5}{2}\right)\left(\frac{24}{2}\right) = 111\,sz\,ft$

Tension

$T = (0.9 - 0.143\,S_{DS})\,D + \rho Q_E/1.4$

- worst case D only has the weight of the walls

$D = \dfrac{17\,psf\,(14.25\,ft \times 5.708\,ft)}{2} = 691.4\,lbs$

$Q_E = T/C\ \text{couple} = \dfrac{1352.9\,lbs\,(14.25\,ft)}{5.708\,ft} = 3377.5\,lbs$

$T = (0.9 - 0.143(1.2))(691.4\,lbs) - (1.3)(3377.5\,lbs)$

$T = 3887.13\,lbs$

Try HD5B w/ (2) 3/4" bolts with a 6x6 chords

Compression

$$C = (1 + 0.143\, S_{DS})D + \cancel{L}^{0} + \rho Q_E/1.4$$

$D = 73\,psf(111\,sqft) + 691.4\,lbs = 8794.4\,lbs$

$Q_E = 3377.5\,lbs$

$C = (1 + 0.143(1.2))(8794.4\,lbs) + (1.3)(3377.5\,lbs)$

$C = 14694.27\,lbs$

→ By Inspection 6x6 is adequate

$\boxed{\text{Use 6x6 DF-L No.1 Chord w/ HD5B w/ (2) 3/4" Bolts}}$

Cantilever Diaphragm

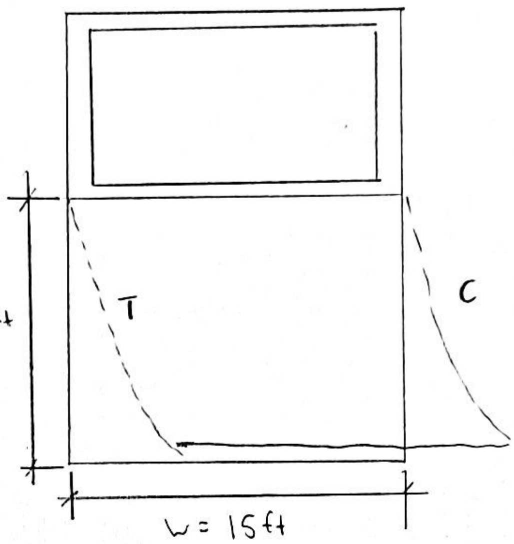
SDPWS §4.2.5.2 Open Front Structures

L/w Ratio $< 1.5:1$

$L < 35$ ft

$L/w = \frac{15.5}{15} = 1.033:1 < 1.5:1$ ✓

$L = 15.5$ ft < 35 ft ✓



Cantilever Chord

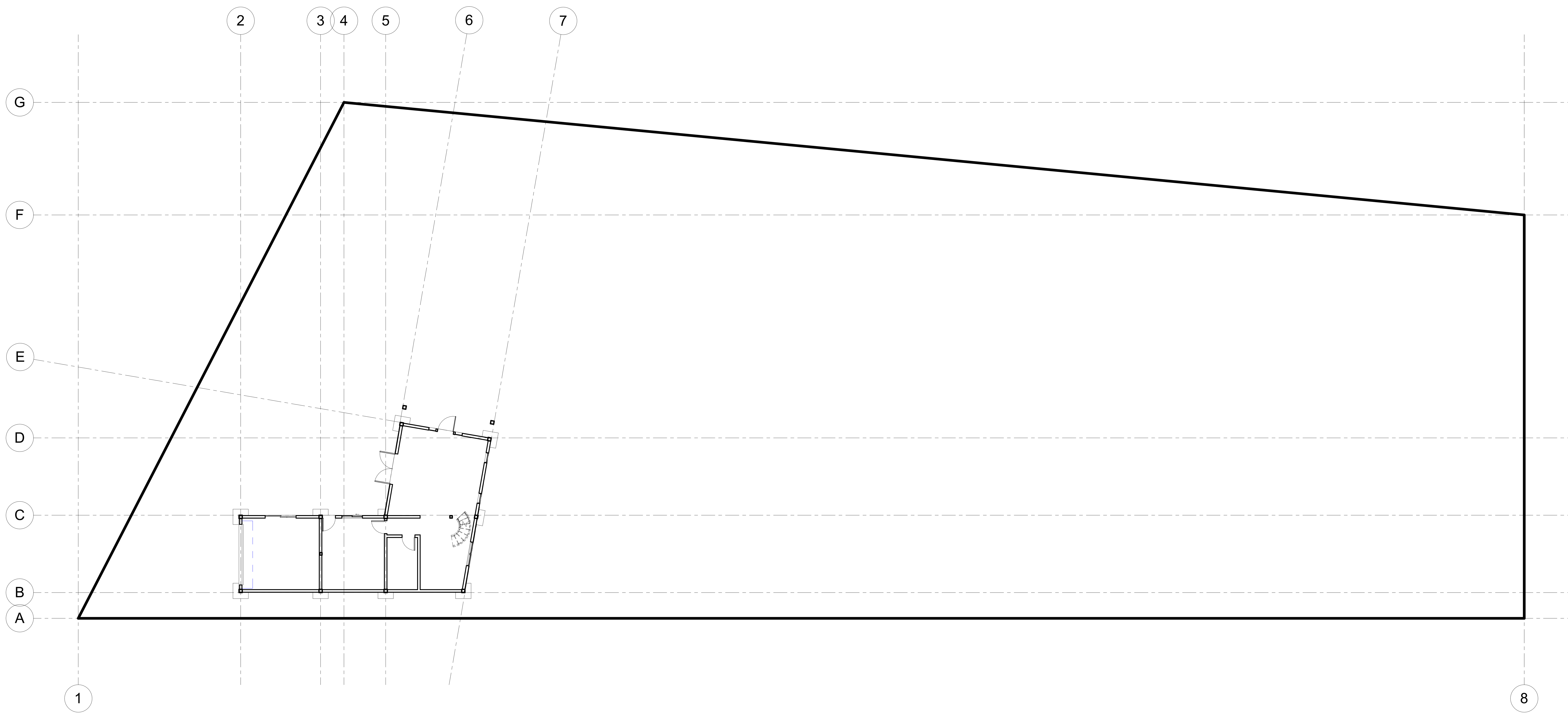
$M = 47.7$ Kft

T/C Couple $= \frac{M}{d} = \frac{47.7 \text{ kft}}{15 \text{ ft}} = 3.18^k$

→ Chord = 5 1/8" × 13 1/2" GLB

→ Less axial Load than collectors (See collector design), adequate by inspection

$\boxed{\text{Use } 5\tfrac{1}{8}" \times 13\tfrac{1}{2}" \text{ GLB for Cantilever Chords}}$



1 SITE VIEW
3/32" = 1'-0"

GUMBY AND CO. STRUCTURAL ENGINEERING

1 Grand Ave. San Luis Obispo CA, 93401

PROJECT INFO:

SENIOR PROJECT

ADVISOR: PROFESSOR JOHN LAWSON

DATE:
6/10/2021 9:51:08 AM

PROJECT:
ACCESSORY DWELLING UNIT

SITE:
1125 EASY ST. MORGAN HILL, CA 95037

REVISIONS

No.	DESC.	DATE

DRAWN BY: Author

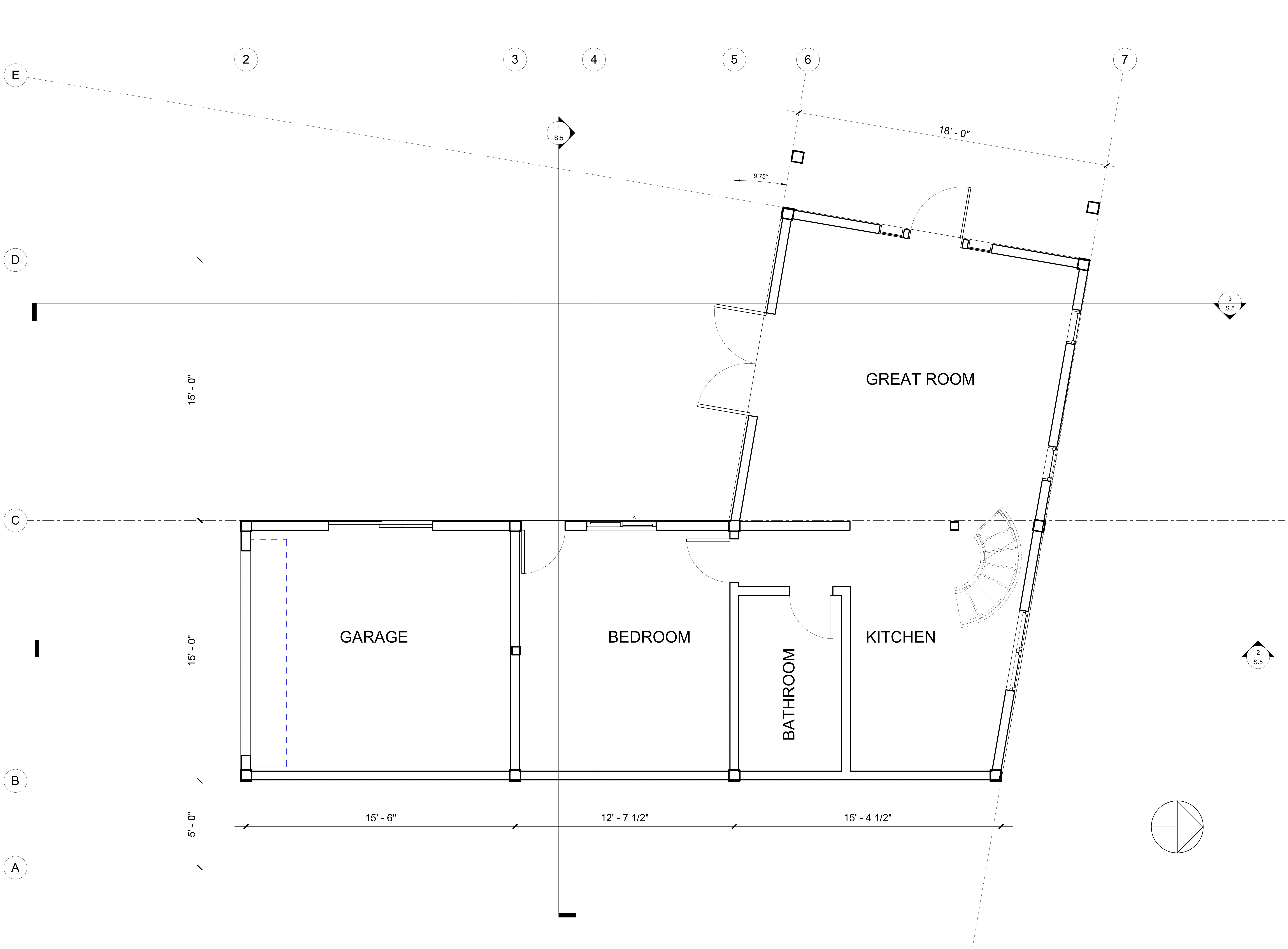
CHECKED BY: Checker

PLOT DATE: 6/10/2021 9:51:08 AM

SHEET NAME:
SITE PLAN

SCALE:
3/32" = 1'-0"

SHEET No.:
A.0



1 Level 2
3/8" = 1'-0"

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REVISIONS

No.	DESC.	DATE

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Author

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PLOT DATE:
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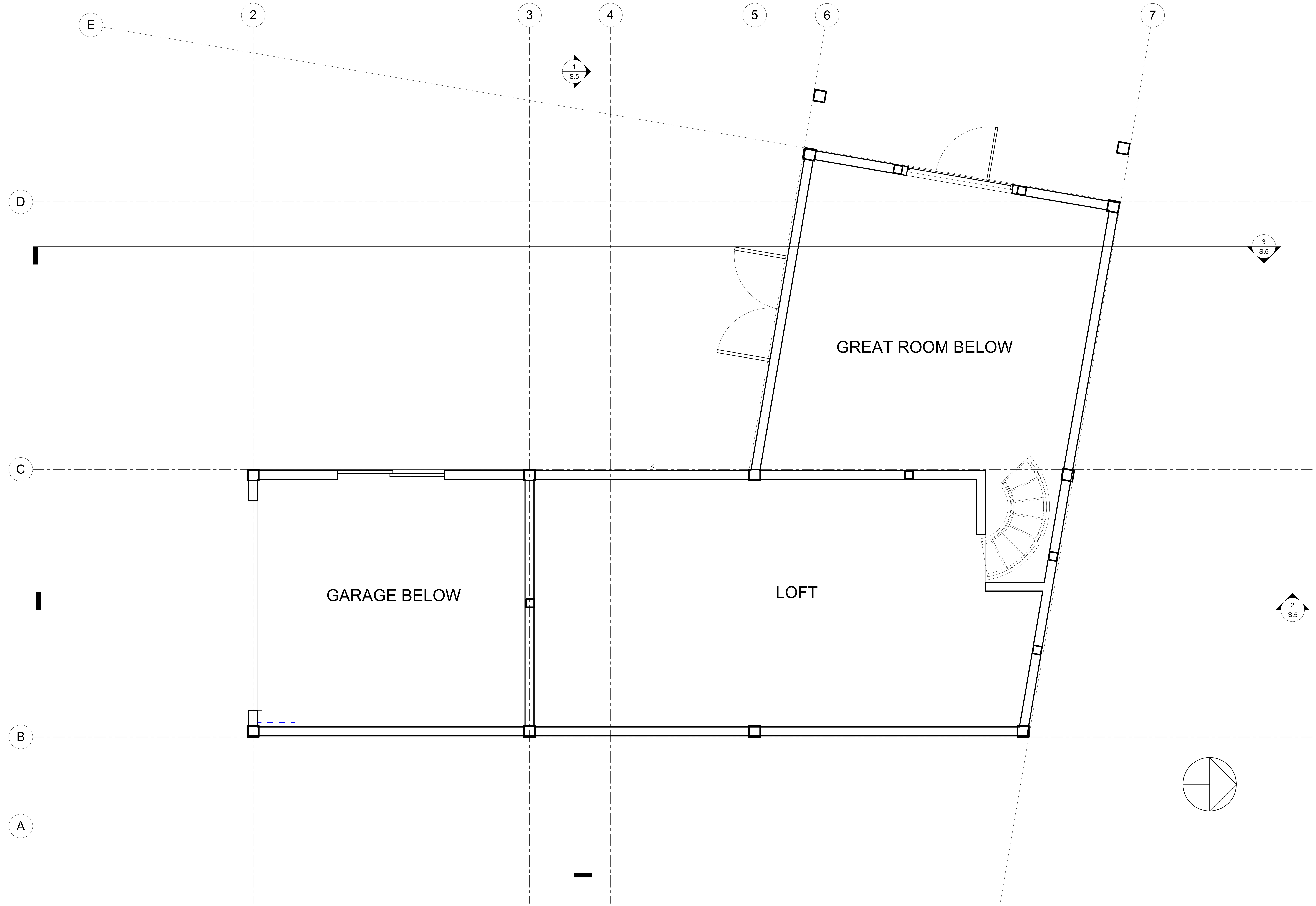
SHEET NAME:

FIRST FLOOR
PLAN

SCALE:
3/8" = 1'-0"

SHEET No.:

A.1



1 TOP OF LOFT
3/8" = 1'-0"

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REVISIONS

No.	DESC.	DATE

DRAWN BY: Author

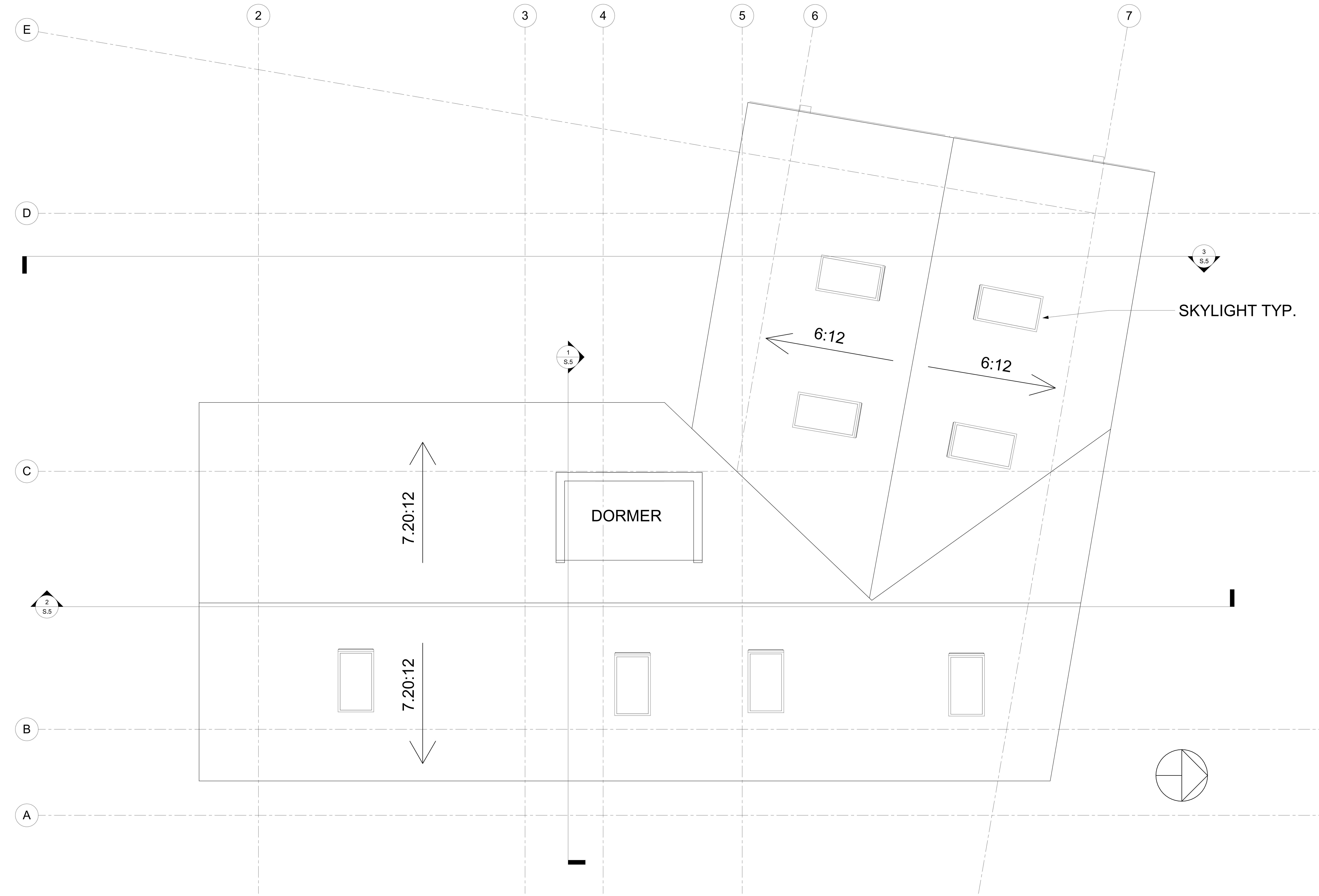
CHECKED BY: Checker

PLOT DATE: 6/10/2021 9:51:08 AM

SHEET NAME: LOFT PLAN

SCALE: 3/8" = 1'-0"

SHEET No.: A.2



1 Level 4
3/8" = 1'-0"

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PLOT DATE: 6/10/2021 9:51:08 AM

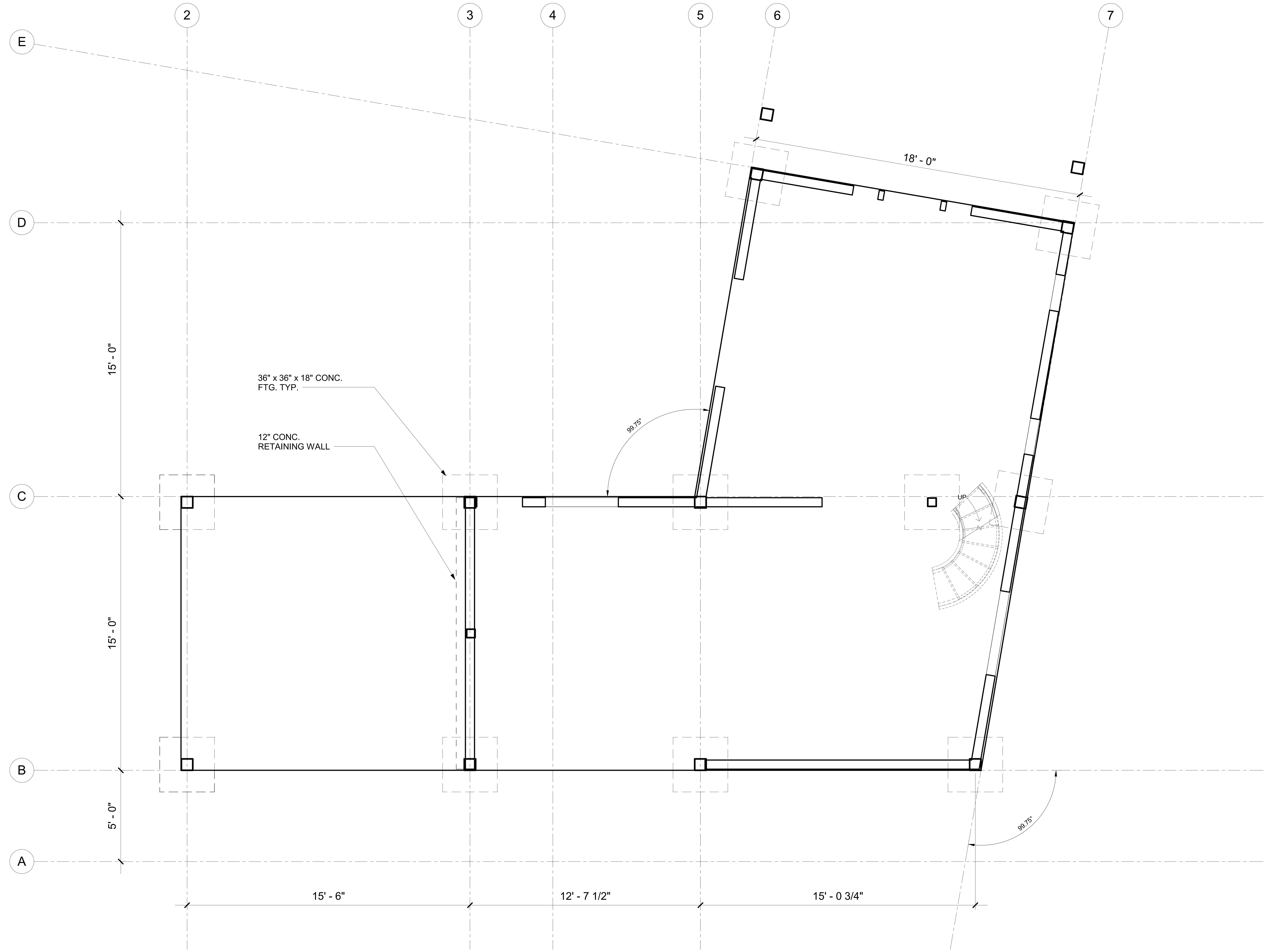
SHEET NAME:

ROOF PLAN

SCALE: 3/8" = 1'-0"

SHEET No.:

A.3



1 Level 2
3/8" = 1'-0"

GUMBY AND CO. STRUCTURAL ENGINEERING

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CA, 93401

PROJECT INFO:

SENIOR PROJECT

ADVISOR: PROFESSOR
JOHN LAWSON

DATE:
6/10/2021 9:51:09 AM

PROJECT:

ACCESSORY
DWELLING UNIT

SITE:

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HILL, CA 95037

REVISIONS

No.	DESC.	DATE

DRAWN BY:
Author

CHECKED BY: Checker

PLOT DATE:
6/10/2021 9:51:09 AM

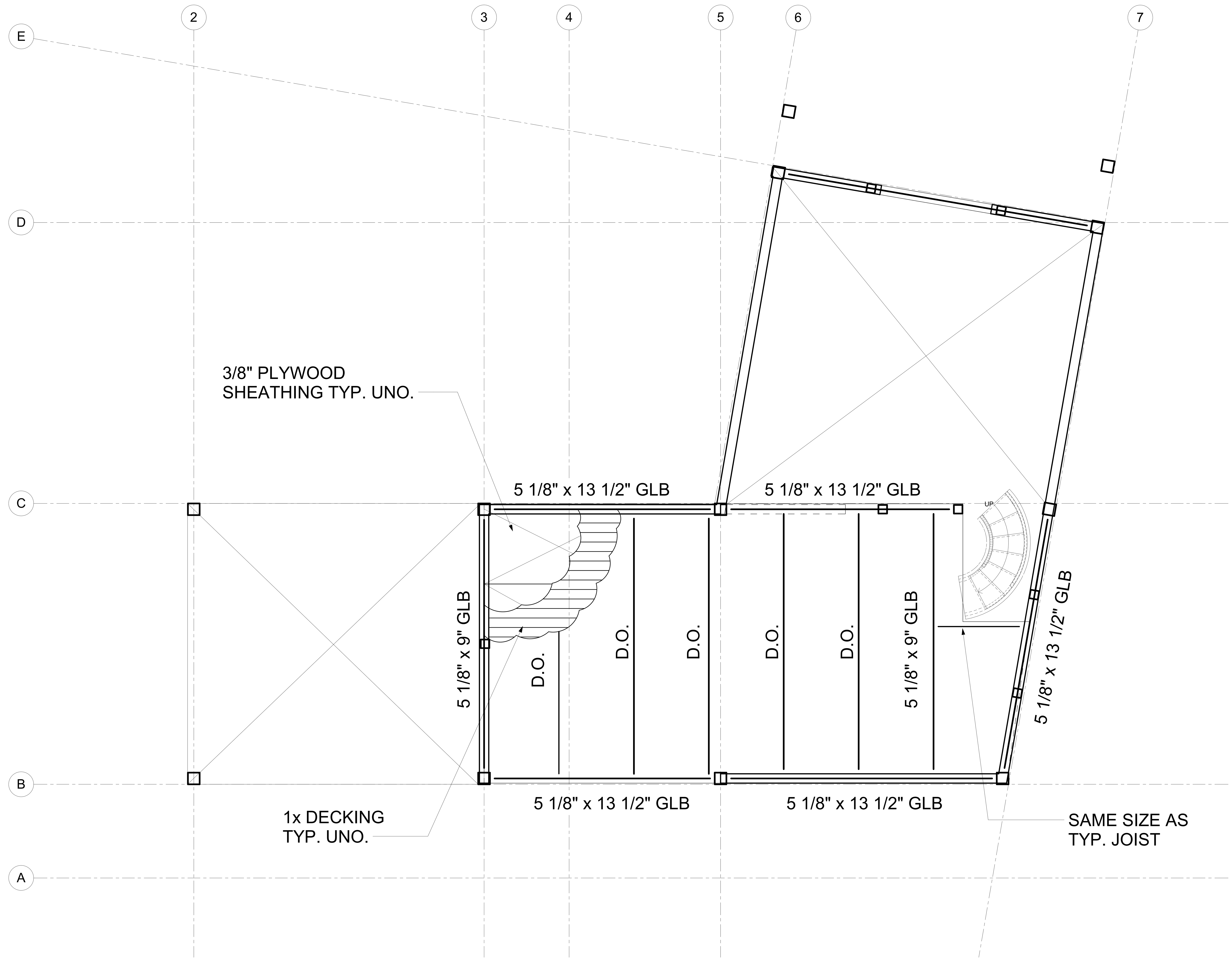
SHEET NAME:

FOUNDATION
PLAN

SCALE:
3/8" = 1'-0"

SHEET No.:

S.1



1 TOP OF LOFT
3/8" = 1'-0"

**GUMBY AND CO.
STRUCTURAL
ENGINEERING**

1 Grand Ave. San Luis Obispo
CA, 93401

PROJECT INFO:

SENIOR PROJECT

ADVISOR: PROFESSOR
JOHN LAWSON

DATE:
6/10/2021 9:51:09 AM

PROJECT:

ACCESSORY
DWELLING UNIT

SITE:

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HILL, CA 95037

REVISIONS

No.	DESC.	DATE

DRAWN BY:
Author

CHECKED BY: Checker

PLOT DATE:
6/10/2021 9:51:09 AM

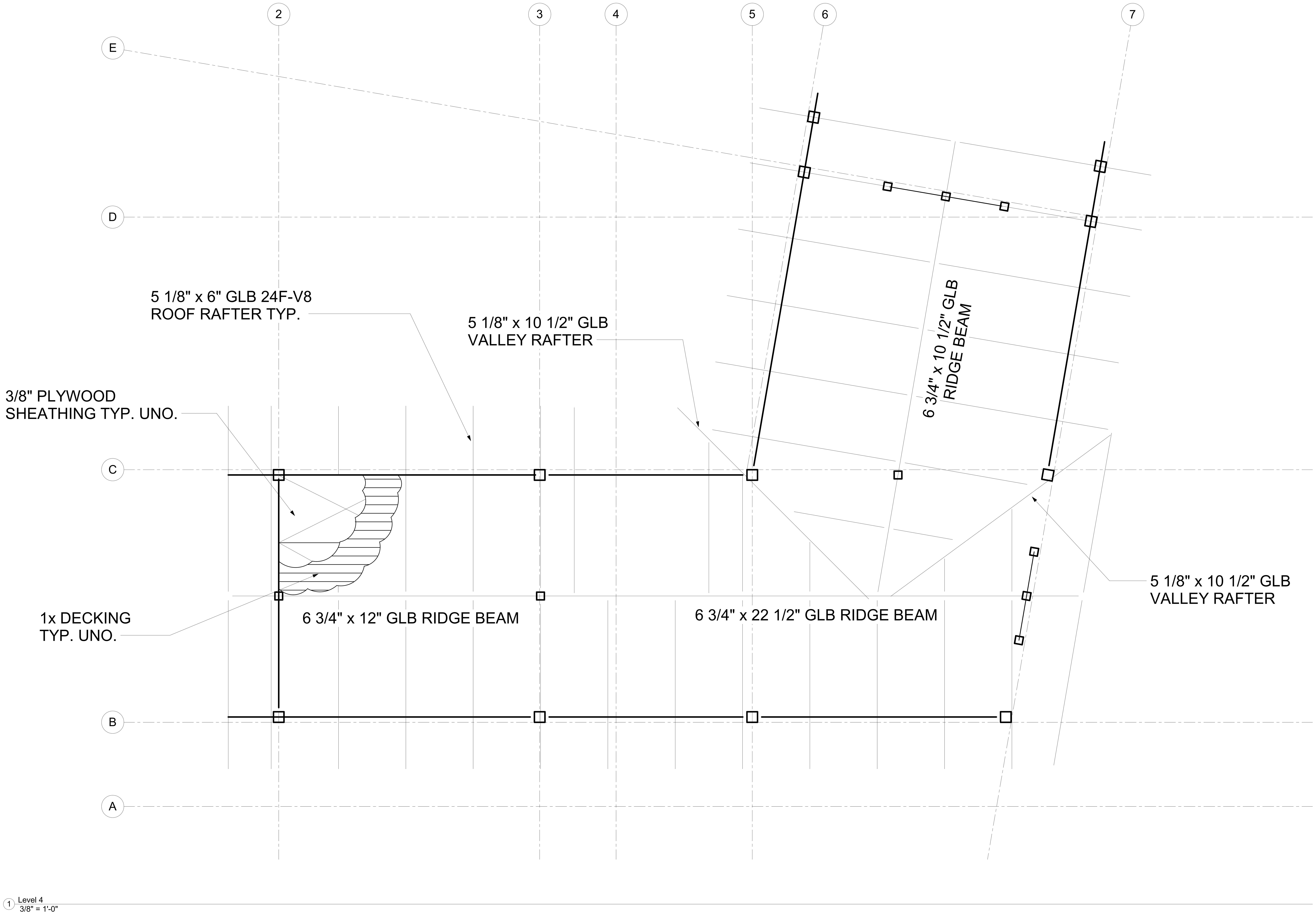
SHEET NAME:

LOFT FRAMING
PLAN

SCALE:
3/8" = 1'-0"

SHEET No.:

S.2



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SENIOR PROJECT

ADVISOR: PROFESSOR JOHN LAWSON

DATE:
6/10/2021 9:51:09 AM

PROJECT:
ACCESSORY DWELLING UNIT

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REVISIONS

No.	DESC.	DATE

DRAWN BY: Author

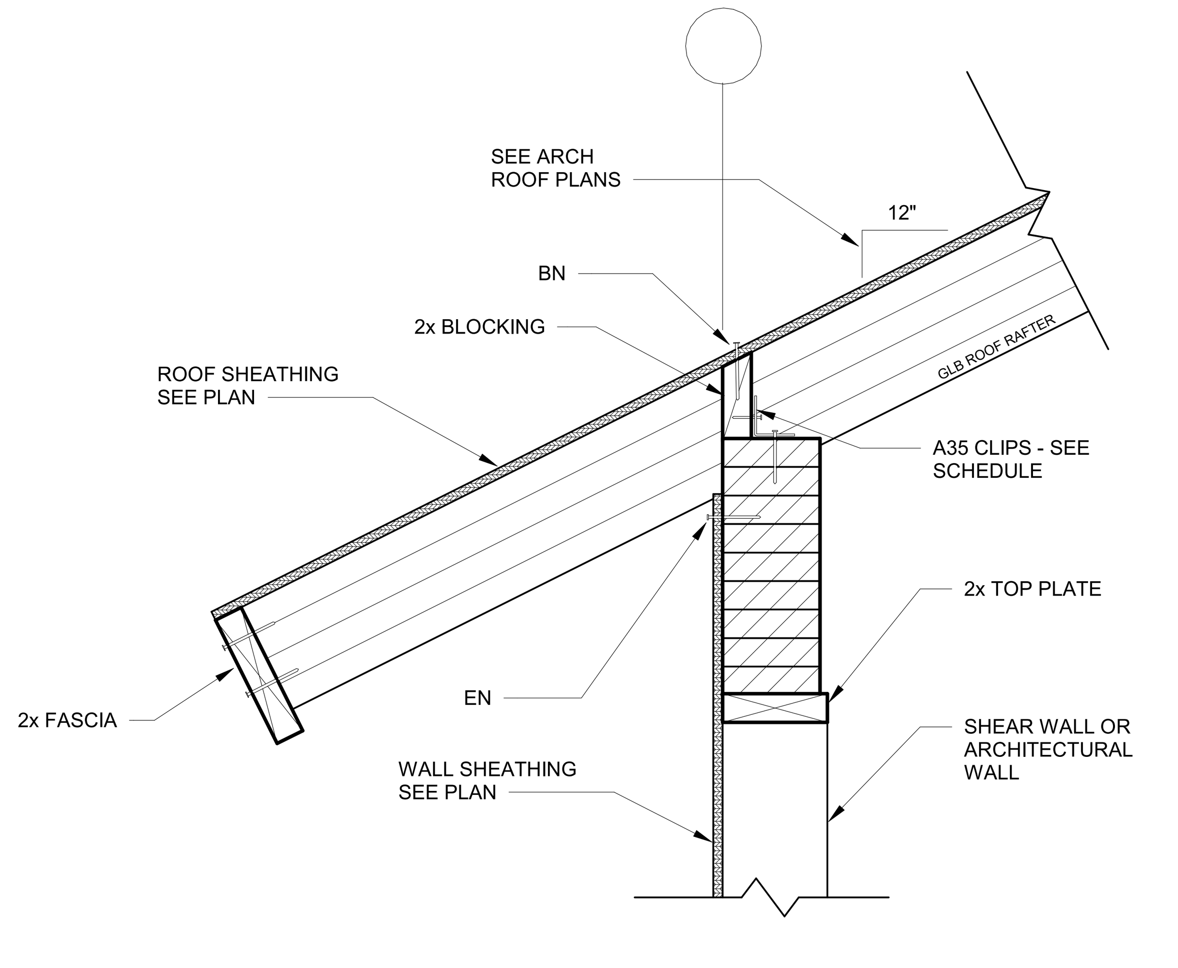
CHECKED BY: Checker

PLOT DATE: 6/10/2021 9:51:09 AM

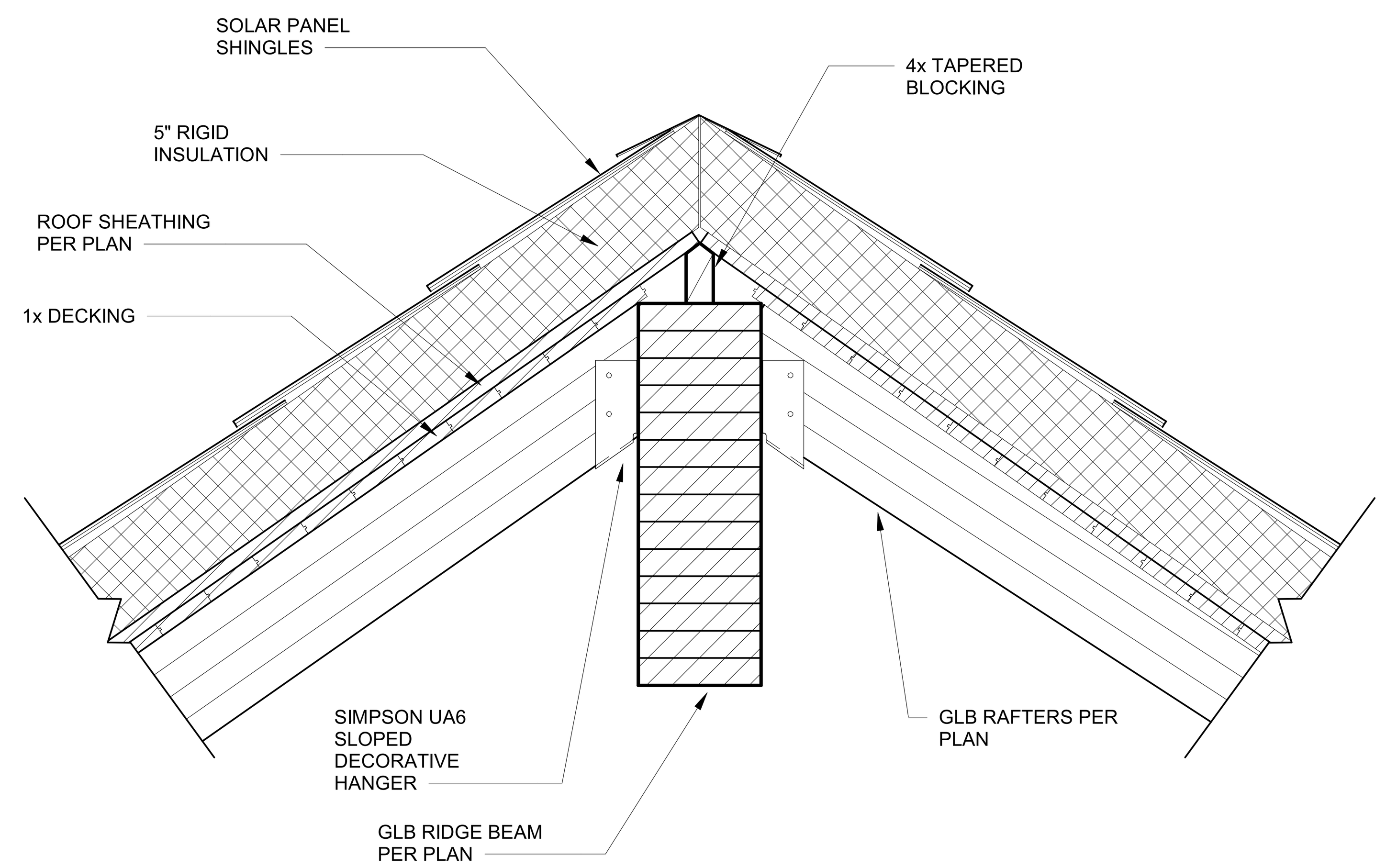
SHEET NAME:
ROOF FRAMING

SCALE: 3/8" = 1'-0"

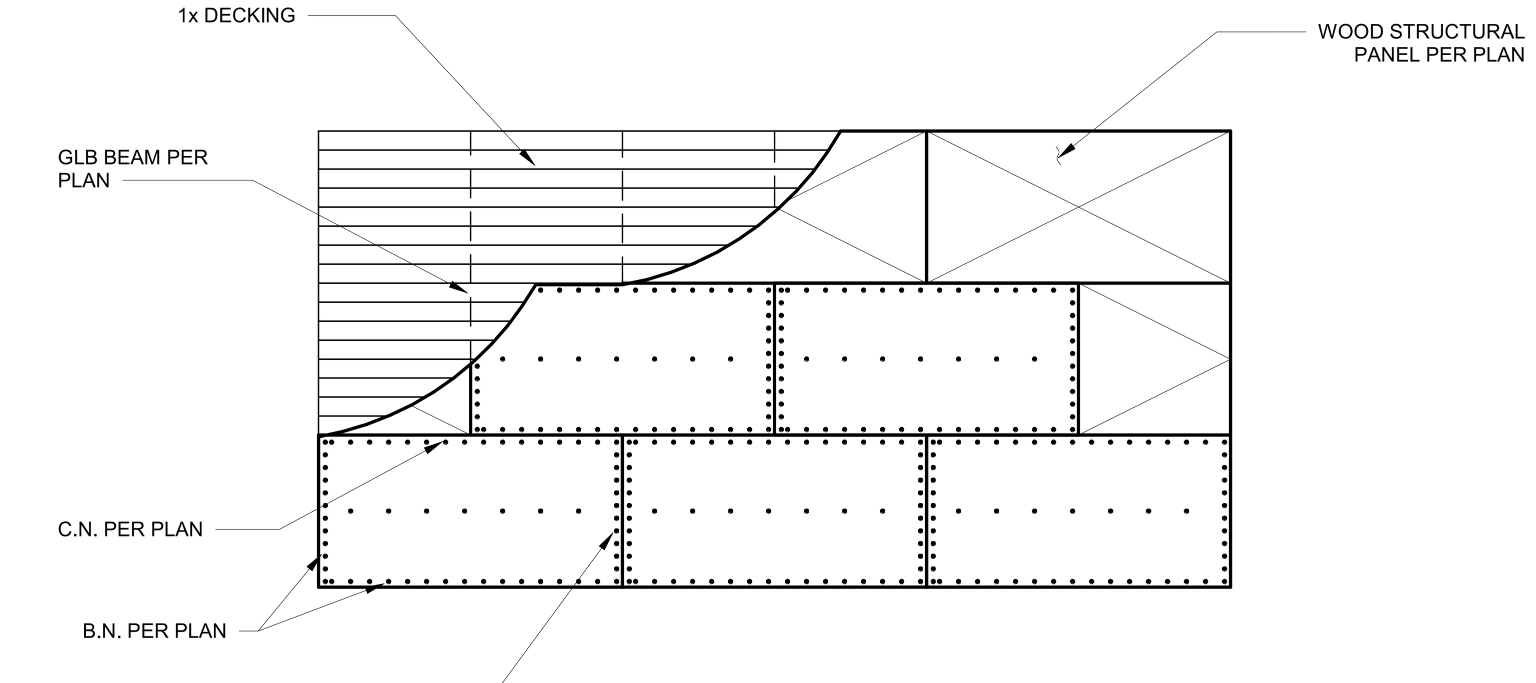
SHEET No.:
S.3



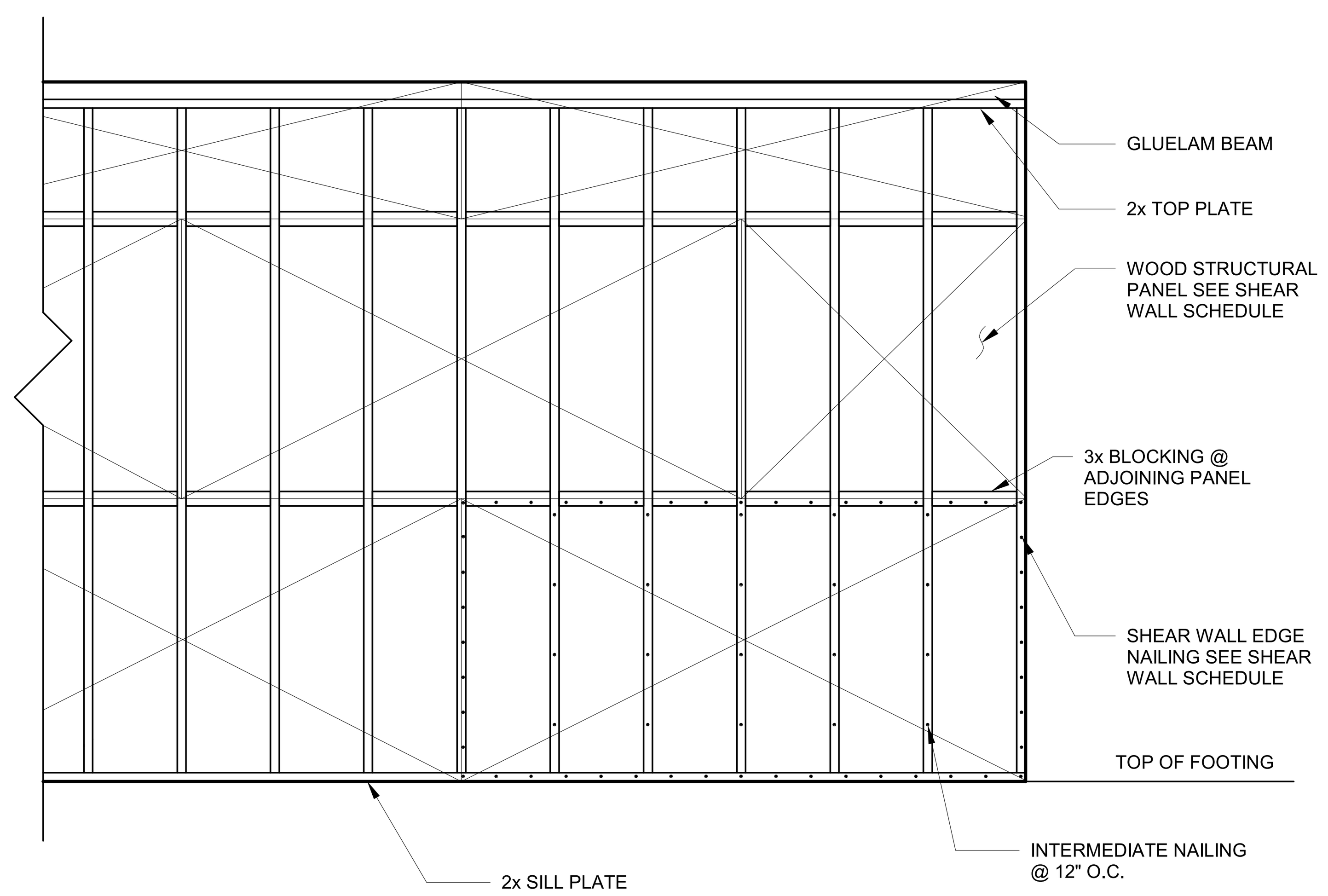
7 ROOF RAFTER TO EXTERIOR WALL
1 1/2" = 1'-0"



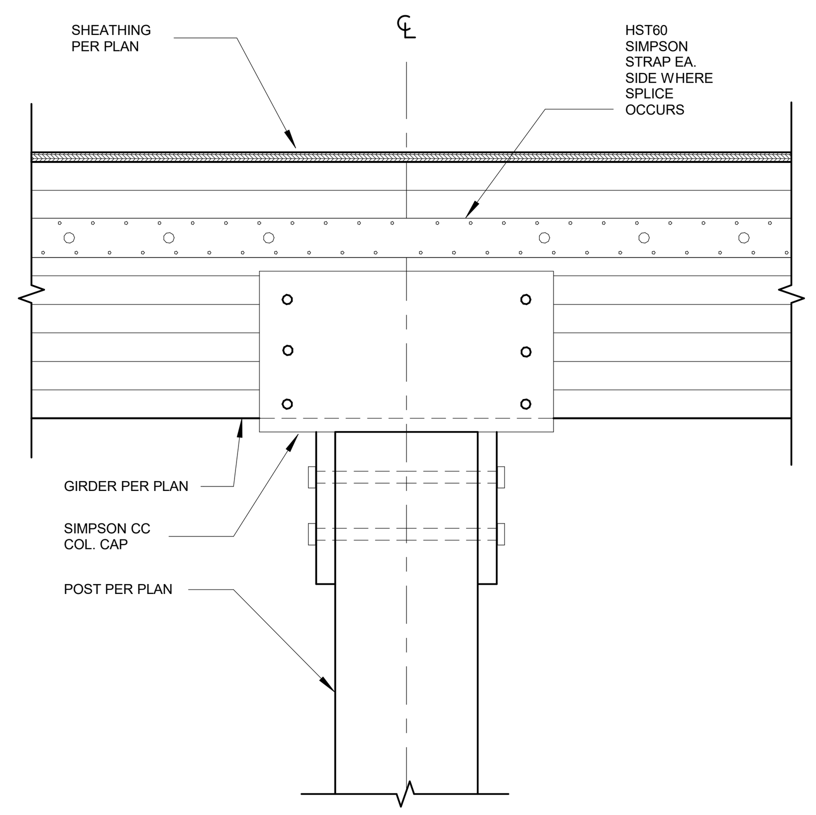
4 ROOF RAFTER TO RIDGE BEAM
1 1/2" = 1'-0"



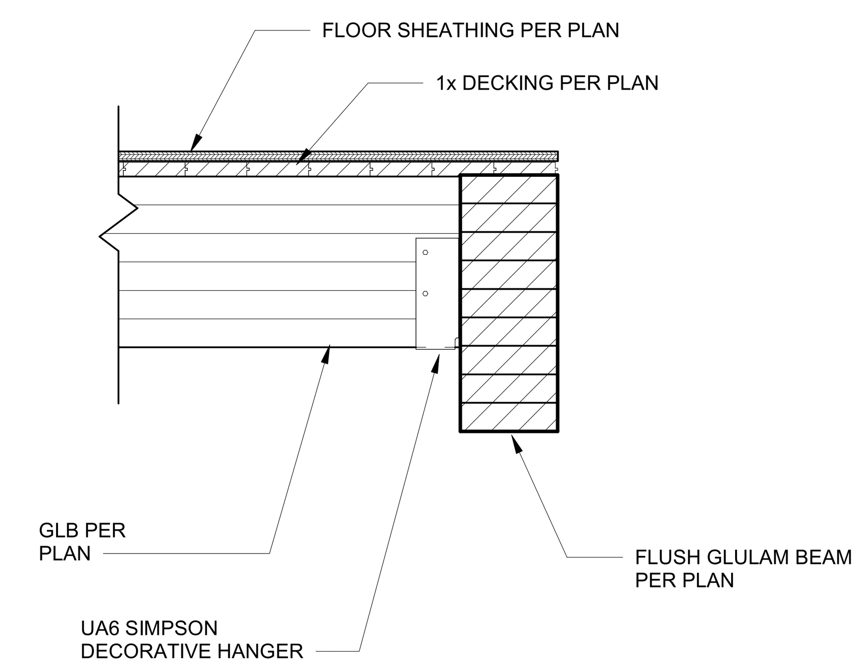
1 TYPICAL DIAPHRAGM NAILING
1/4" = 1'-0"



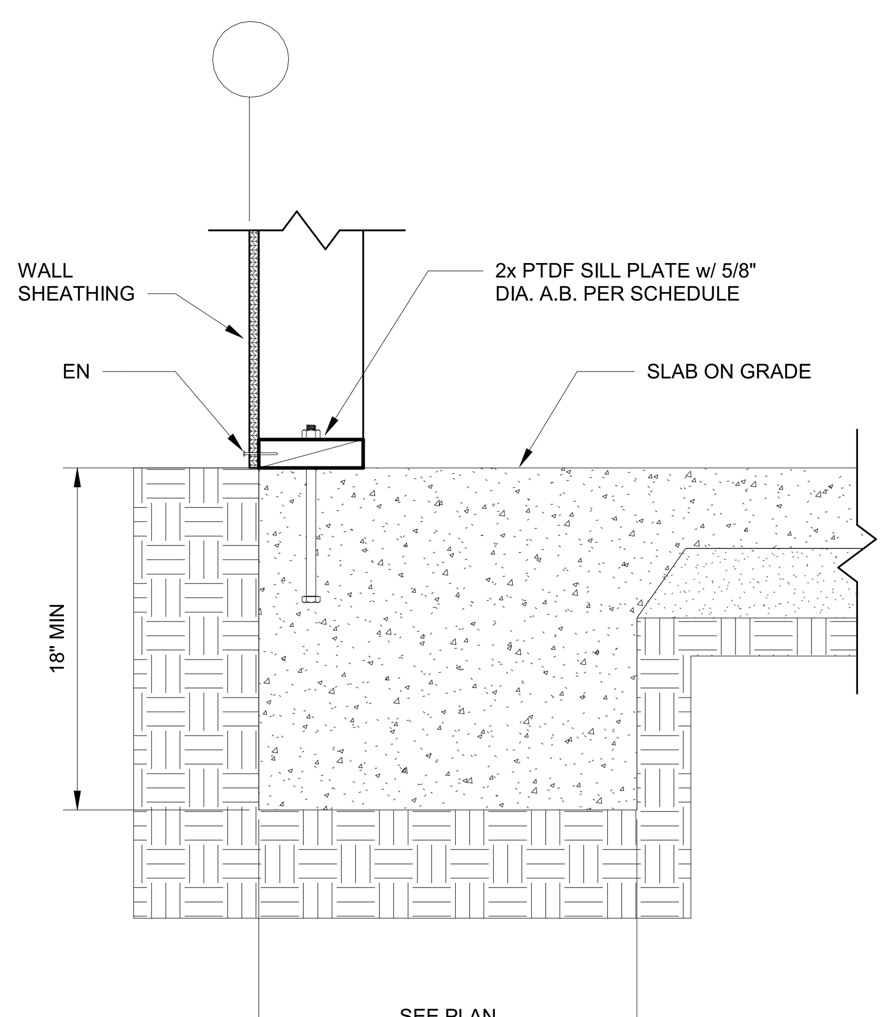
8 TYPICAL SHEAR WALL DETAIL
1/2" = 1'-0"



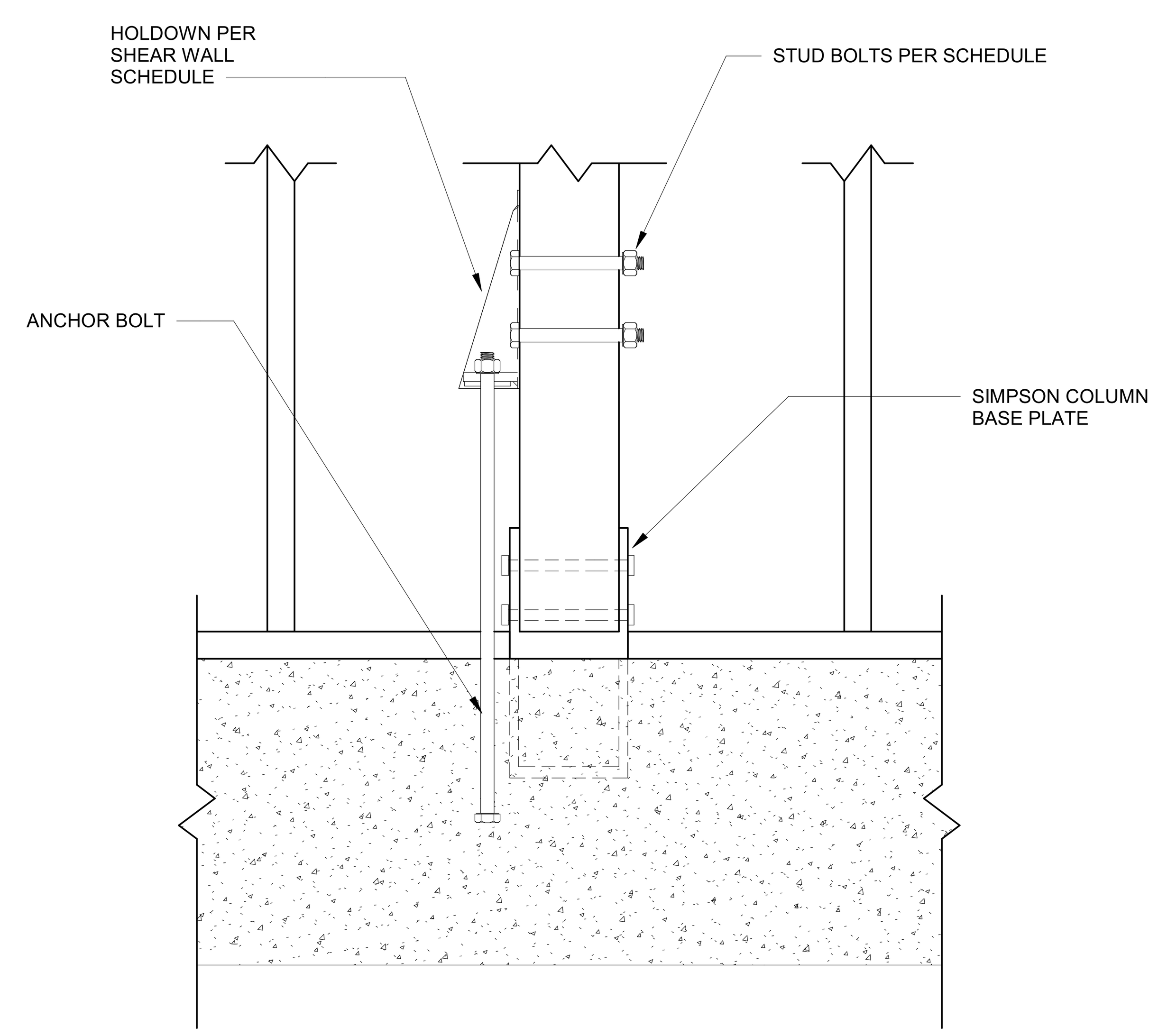
5 GIRDER TO POST
1 1/2" = 1'-0"



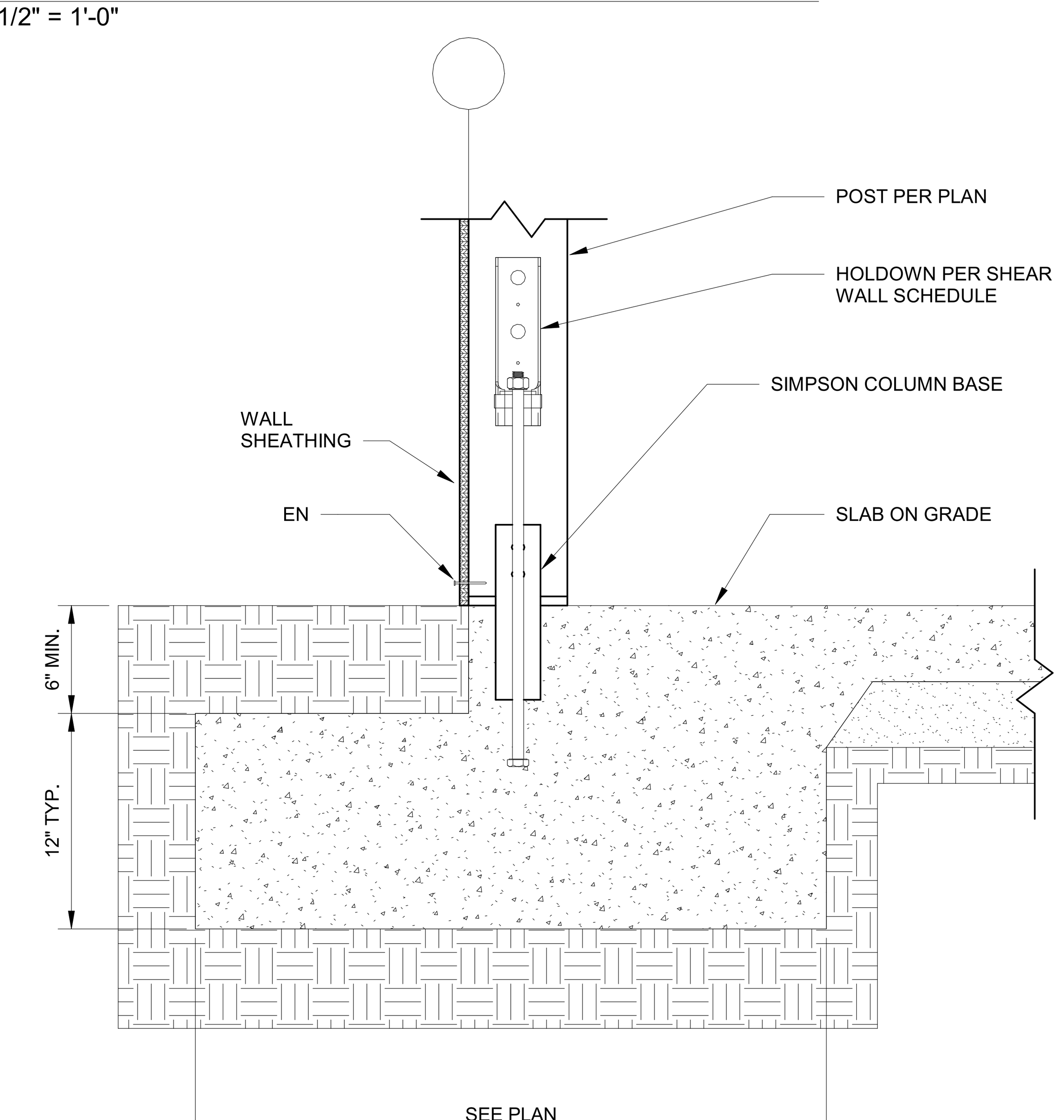
2 LOFT BEAM TO GIRDER
1 1/2" = 1'-0"



9 SHEAR WALL TO EDGE OF FOUNDATION
1 1/2" = 1'-0"



6 HOLDOWN TO FOUNDATION DETAIL
1 1/2" = 1'-0"



3 POST TO EDGE OF FOUNDATION
1 1/2" = 1'-0"

GUMBY AND CO. STRUCTURAL ENGINEERING

1 Grand Ave. San Luis Obispo
CA, 93401

PROJECT INFO:

SENIOR PROJECT

ADVISOR: PROFESSOR
JOHN LAWSON

DATE:
6/10/2021 9:51:09 AM

PROJECT:

ACCESSORY
DWELLING UNIT

SITE:

1125 EASY ST. MORGAN
HILL, CA 95037

REVISIONS

No.	DESC.	DATE

DRAWN BY: Author

CHECKED BY: Checker

PLOT DATE: 6/10/2021 9:51:09 AM

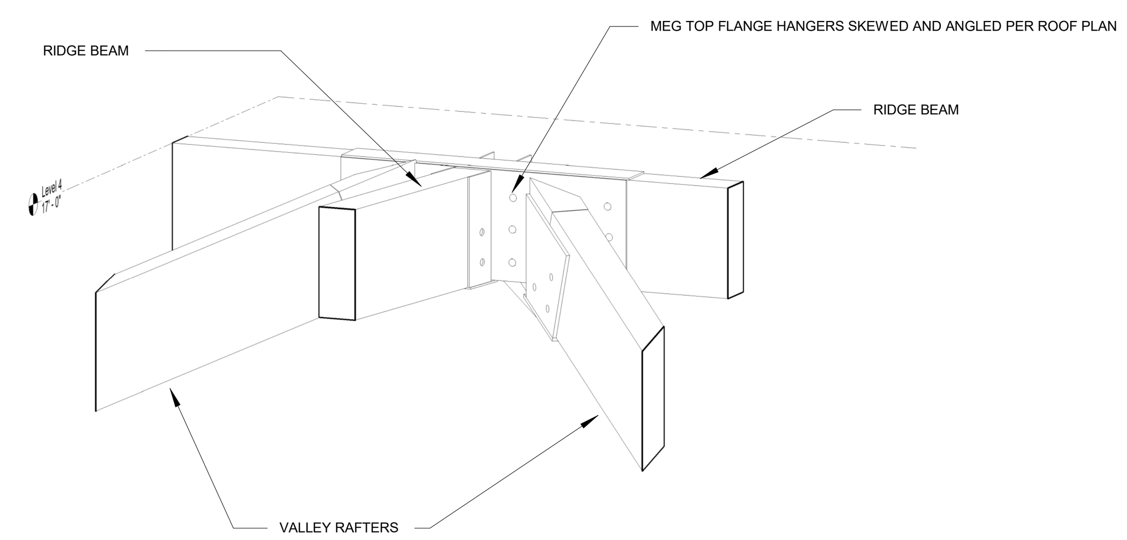
SHEET NAME:

TYPICAL
DETAILS

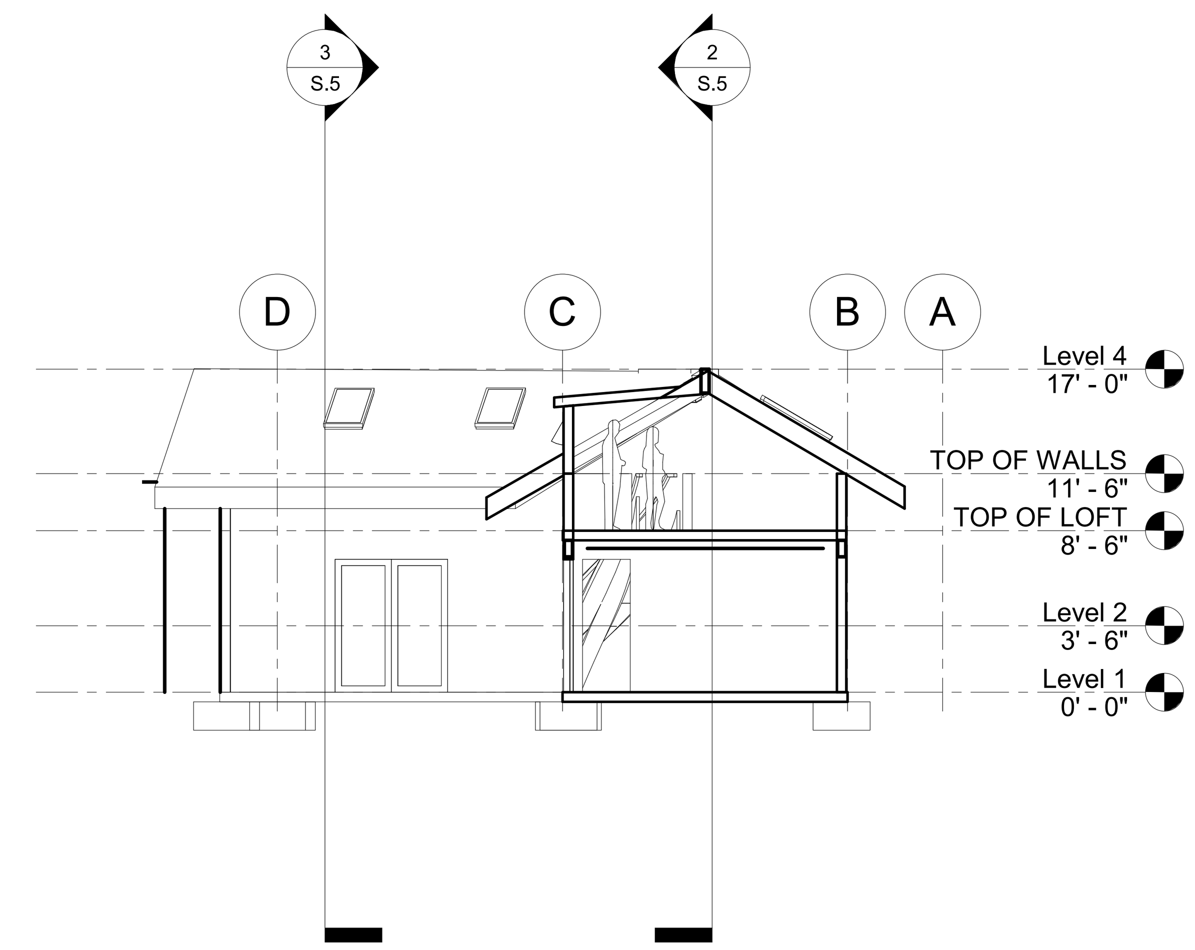
SCALE: As indicated

SHEET No.:

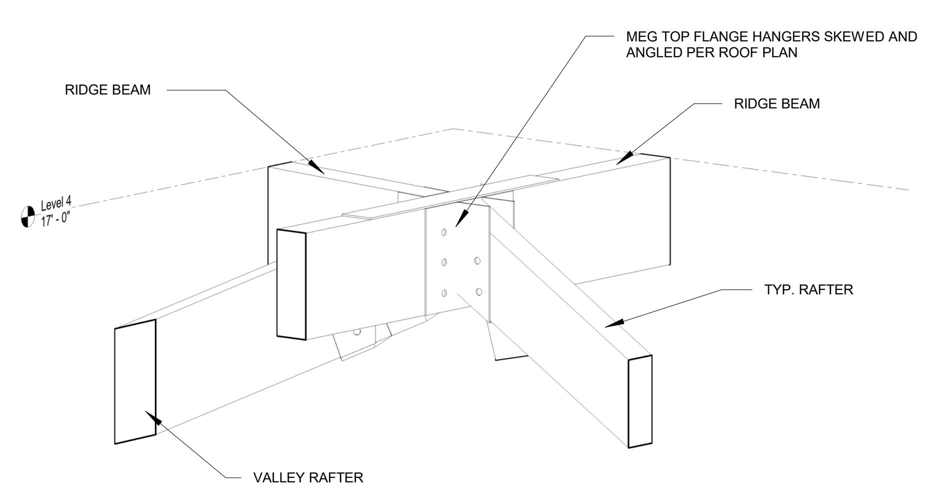
S.4



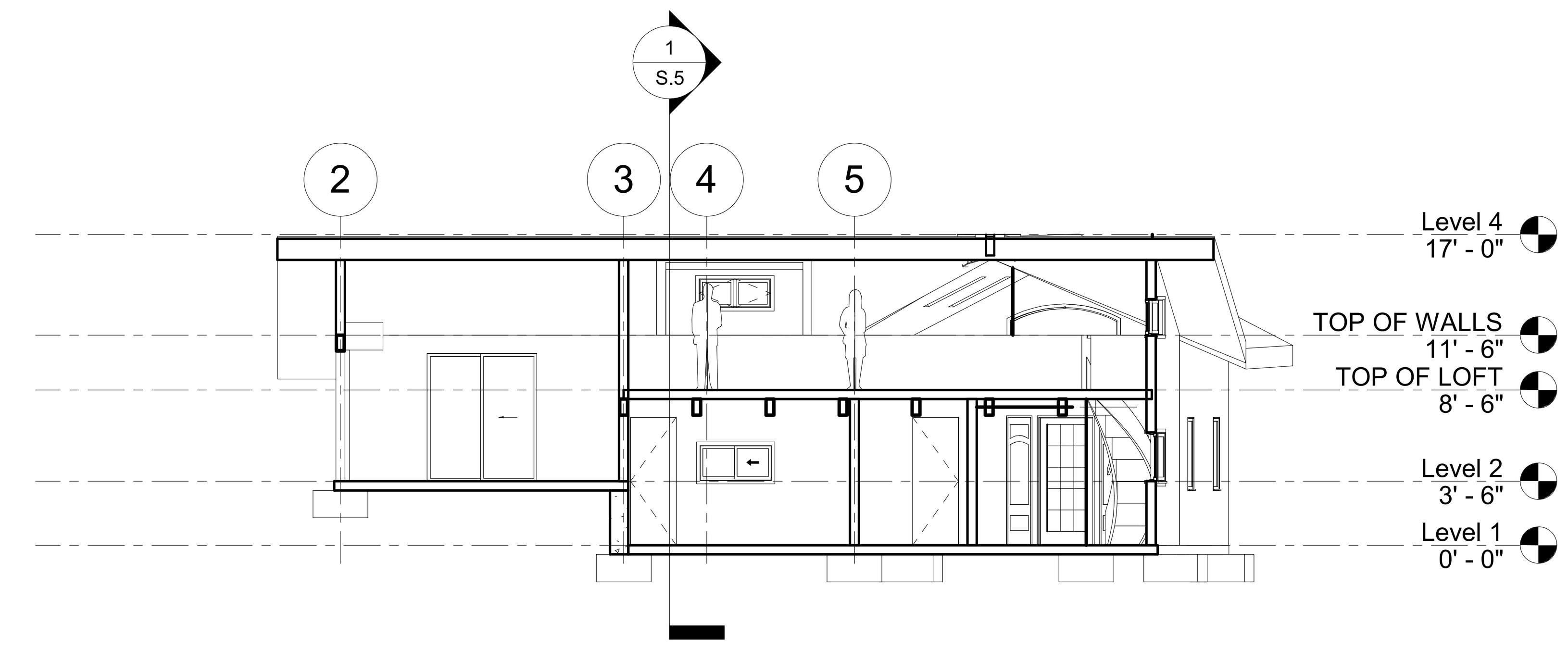
4 RIDGE BEAM CONNECTION LOOKING EAST
1 1/2" = 1'-0"



1 Section 1
1/8" = 1'-0"

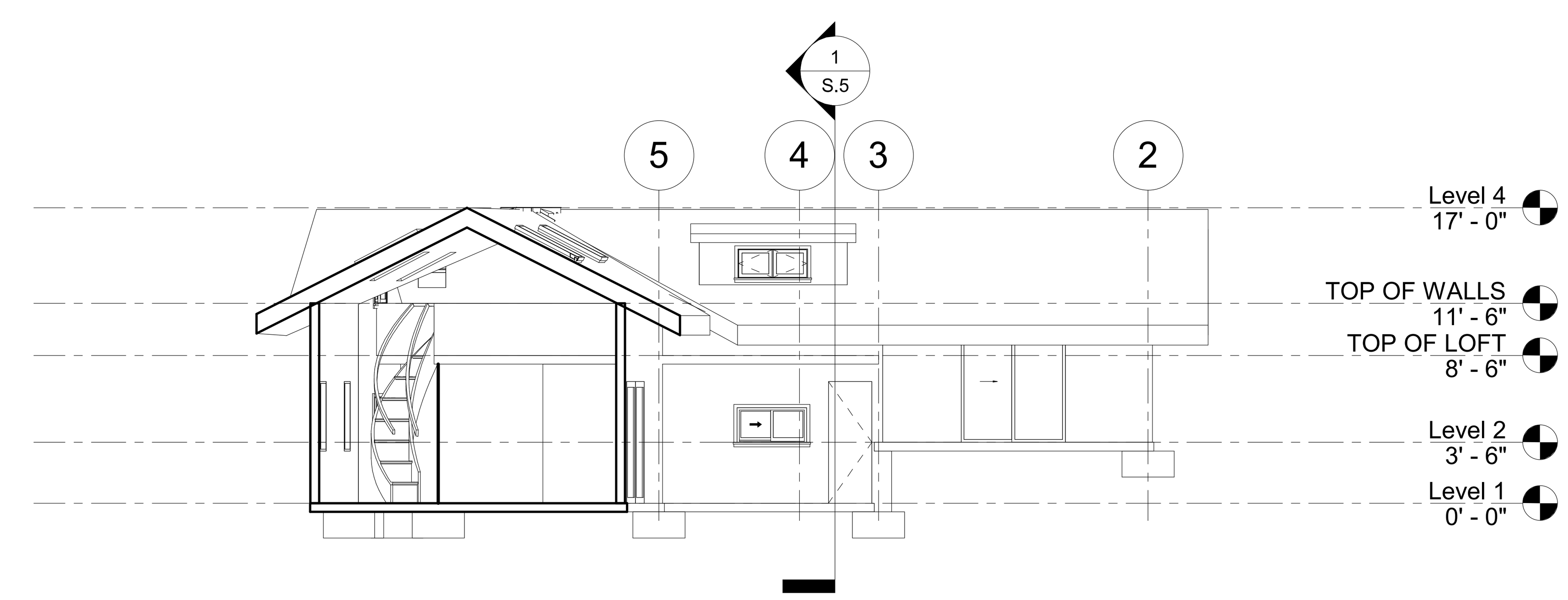


5 RIDGE BEAM CONNECTION LOOKING WEST
1 1/2" = 1'-0"



2 Section 2
1/8" = 1'-0"

6 3D VIEW



3 Section 3
1/8" = 1'-0"

GUMBY AND CO. STRUCTURAL ENGINEERING

1 Grand Ave. San Luis Obispo CA, 93401

PROJECT INFO:

SENIOR PROJECT

ADVISOR: PROFESSOR JOHN LAWSON

DATE:

6/10/2021 9:51:11 AM

PROJECT:

ACCESSORY DWELLING UNIT

SITE:

1125 EASY ST. MORGAN HILL, CA 95037

REVISIONS

No.	DESC.	DATE

DRAWN BY: Author

CHECKED BY: Checker

PLOT DATE: 6/10/2021 9:51:11 AM

SHEET NAME: SECTION & 3D VIEWS

SCALE: As indicated

SHEET No.: S.5

Nick Scalzo

John Lawson

6/10/21

Senior Project Discussion

My senior project is an accessory dwelling unit for my family in which I did most of the architectural and structural design. Throughout the project I encountered many issues that I had not seen before and set this project apart from a design lab. My first major issue was doing the architecture. I have worked with architects before, but doing the work was a different challenge. I had to go to the city and look through city codes to get permitting requirements as well as limits on building size, height, and placement. The only thing that I had to start off with was an idea from the owner (my parents) and the site dimensions. There was lots of discussion and debate on layout of the rooms and what the owner wanted out of the structure. From there the form of the building started to naturally shape itself. The building took an iterative design process to get to the final architectural form. I started with very tall walls to try and make the loft space as comfortable to walk in as possible, but with limits on having a habitable second floor the roof ended up being heavily sloped and a dormer was added to make the interior height feel larger. The other major change in the shape of the building happened when I made a site visit to lay out the plan of the building and I did not like the way it looked in the space. It taught me a lot about how looking at a Revit model will not tell you how it will look on the site no matter how meticulously the model was crafted. Going out to the site added some very important context that the model could not give me. I ended up angling the front of the building about 10 degrees over to make it fit the space better. The building was only shifted over a couple feet, but it is incredible to see how big of a difference that made in a building of this size. The architecture was an interesting balance of the utility required, aesthetic required, and city requirements that made for a much more difficult process than I had anticipated. I had only ever experienced first year studio with no building design whatsoever, so this gave me a huge respect for what architects do that is way beyond just art. Another set of issues I ran into were some structural problems. Part of the building was angled which led to some very tall, thin, and angled shear walls where the design started to become ridiculous in comparison to every other shear wall in the building. Another unique issue was the cantilevered diaphragm by the garage. There I learned about the requirements that limit these types of diaphragms but also got a glimpse into semi-rigid analysis and a more accurate way to look at diaphragms. All along the way John taught me a lot about what is common in the field. I would often bring a design for something that to John would look very out of place and not common in construction. He guided me to solutions that are both efficient while also being in common practice and thus easier to build. The lessons I learned in this project are something that I will carry to my professional career and onward. I have such a better understanding of how architecture and structure must work together and how constructability also must play a role in all of it. Problem solving on my own and learning from John developed my engineering sense to be able to tackle unknown problems with much more confidence now.

Every project is influenced by global, economic, environmental, social, and cultural issues. Starting at the most local issue, socially accessory dwelling units were created in the code as a form of affordable housing that could be built on lots will extra space instead of expanding neighborhoods and absorbing open land. This project is not meant as an affordable housing option but does still have social roots. The owners want it to house kids, grandkids, or grandparents. It is a structure that has social purpose, and its design was directly influenced by what its social purpose was. Another part of the social issues that have just come up is COVID-19. With the pandemic, in person interaction was very limited and thus traveling to the site was difficult. All processes and meetings took place over the internet. Getting used to dealing with the inherent social barriers that the internet provides will be very important in the future as I believe that this social issue is here to stay with or without COVID.

Another part of the social aspect is a cultural one. My family is a very traditional Italian household and family is everything. Doing any and everything family is what is expected, so having extra space for family members to stay or live in is engrained in my family's culture. In terms of "culture" in Morgan Hill the city has put strict limits on new construction as Morgan Hill is meant to be a small-town branching from San Jose with lots of farmland. Lots of bills have been passed to limit new buildings and protect farmland in the town driven by those who live here. This has influenced the zoning and size requirements for ADUs as making something that is as unobtrusive as possible is important.

Economically the building is not being designed to be as cheap as possible. The system is a post and beam structure which will cost more than typical stud framed building. With this said this was done purposefully to get the look that the owners wanted as well as provide for flexibility of the structure as most of the walls are purely architectural and can be changed. Glulam beams are very efficient in terms of how large they must be compared to the load they will take. This allows a lot less wood to be used which will save some cost. The largest part of the cost of the building will come from the architectural finishes and because the structure was meant to be exposed there will be a lot of finishes that will be avoided. This will also save some of the cost. Tied into environmental and economical, the roof was designed to carry solar shingles which may provide power for the house and nearby outdoor lights. It will be an initial cost, but overtime will save money.

Environmentally this building is currently not designed to meet any advanced LEED certification, but may in the future. Glulam beams benefit the environment because they are constructed from extraneous layers of wood and glued together. It uses scraps in layers in the middle of a member that do not take large stresses. This means that in order to get a large beam the manufacturers do not have to cut down a 100-year-old tree to get a massive timber beam, but instead can use smaller more sustainably farmed wood to create the same size. As talked about above there is a plan to incorporate solar shingled into the building which creates a renewable energy source for the building. Other environmental concerns fall onto appliances and mechanical equipment that will be chosen at a later time.

Lastly are global issues, which currently the biggest effect on this project is COVID. COVID has affected every process in creating a building including the design, permitting, and

construction. All the building planning is online and so making sure that communication is clear is important. Plywood prices right now are incredibly expensive due to COVID, so not building it right now will save money but also designing using thinner plywood will save money. Globally this structure will not have much affect besides its environmental impact. I think that prioritizing the environment in all projects no matter how small can start to set examples and continue to shift of the field globally toward prioritizing environmentally friendly buildings.