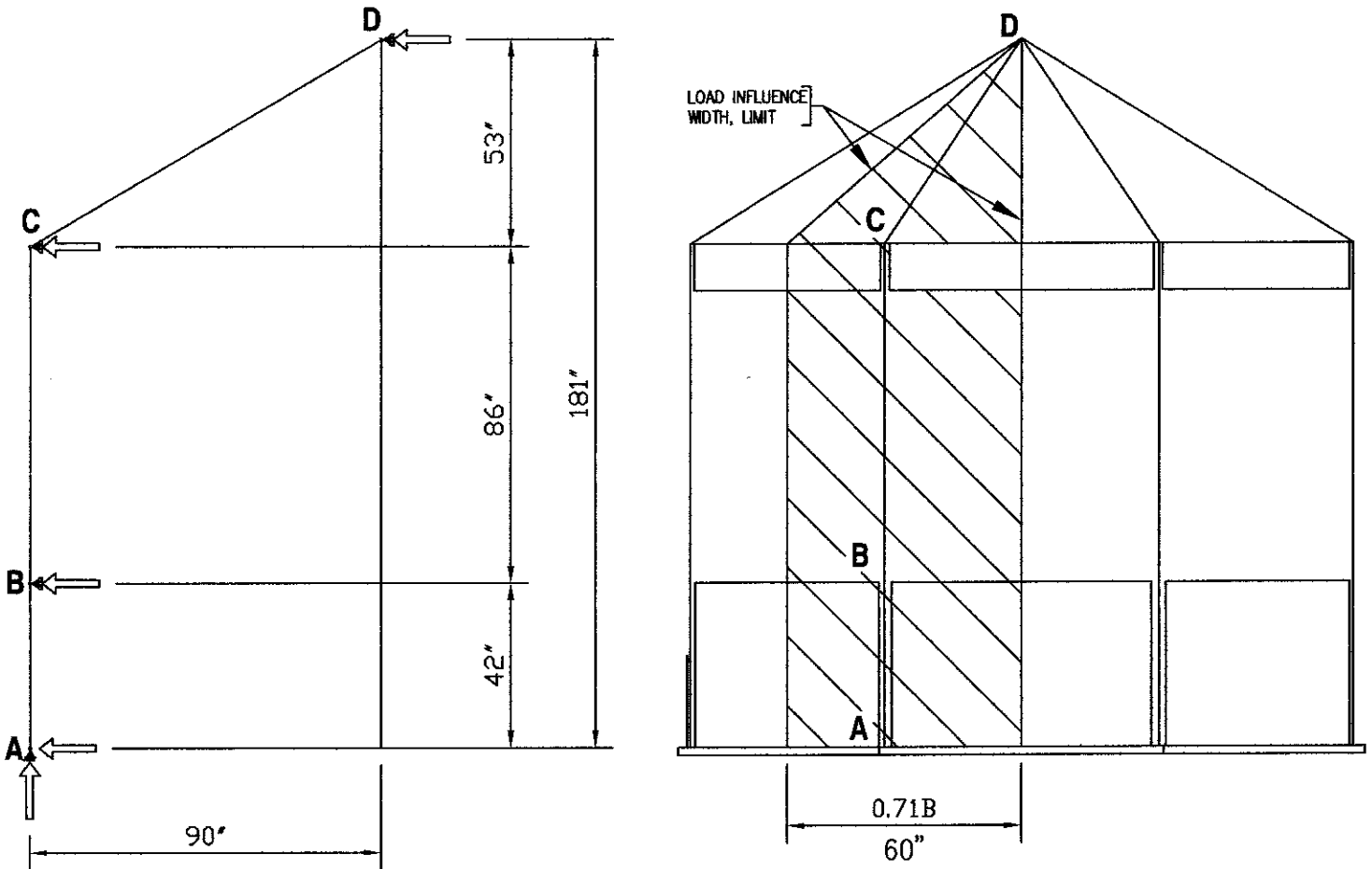


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STRUCTURAL ANALYSIS VIXEN HILL MODEL 15' GAZEBO



Toll Brothers, Inc.
FORKED RIVER, NJ



CLIENT	Toll Brothers, Inc.
PROJECT	Forked River, New Jersey
JOB No:	12209
CALCULATED BY	Robert W. Riiber, P.E.
CHECKED BY	Graham Henderson, P.E., L.S.
STRUCTURE	Vixen Hill - 15' Gazebo

CALCULATION INDEX

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8. Additional Overturning Stability.
9. Column Base Strap Design.

1.1 Useful Units Defined

$$\begin{aligned}
 \text{plf} &:= \frac{\text{lb}}{\text{ft}} & \text{psf} &:= \frac{\text{lb}}{\text{ft}^2} & \text{pcf} &:= \frac{\text{lb}}{\text{ft}^3} & \text{kip} &:= 1000\text{-lb} & \text{ksi} &:= \frac{\text{kip}}{\text{in}^2} & \text{ksf} &:= \frac{\text{kip}}{\text{ft}^2}
 \end{aligned}$$

2.1 MATERIALS

Steel - A36	Yield Stress	$F_{yA36} := 36\text{-ksi}$	Modulus of Elasticity	$E := 29000\text{-ksi}$
	Ultimate stress	$F_{uA36} := 58\text{-ksi}$		
	Density	$\rho_{A36} := 490\text{-pcf}$		

Wood

Select Structural Western Cedar

Bending Stress	$F_{bw} := 1000\text{-psi}$	Modulus of Elasticity	$E' := 1000\text{-ksi}$
Shear Strength	$F'_v := 75\text{-psi}$	Stress Perp. to Grain	$F_{cL} := 425\text{-psi}$

Density

Assumed Water Content $mc := 22\%$

Specific Gravity $G_{\text{WW}} := 0.36$

$$\rho_{\text{cedar}} := 62.4\text{-pcf} \cdot \left(\frac{G}{1 + G \cdot 0.9 \cdot mc} \right) \cdot (1 + mc) \quad \rho_{\text{cedar}} = 25.6\text{ pcf}$$

Concrete	Compressive Strength	$f_c := 3500\text{-psi}$	Modulus of Elasticity	$E_c := 57000 \cdot \sqrt{\frac{f_c}{\text{psi}}}\text{-psi}$ $E_c = 3.4 \times 10^6\text{ psi}$
	Reinforcing Steel Yield stress	$F_{yc} := 60\text{-ksi}$		
	Density :	$\rho_{\text{con}} := 150\text{-pcf}$		

2.2 CODES

IBC-2009	International Building Code
UCC-2009	NJ Uniform Construction Code
AISC	Steel Construction Manual ASD
ACI - 318	Building Code for Requirements for Structural Concrete
ASCE 7-05	Minimum Design Loads for Buildings w/NJ modifications
NDS	National Design Specification for Wood Construction with Supplement.

3.1 Method 2, Analytical procedure, Design Wind Pressure - Main Wind Force Resisting System

Horizontal building dimension measured normal to wind direction $B := 21\text{-ft}$
 Structure overall height $h := 16\text{-ft}$
 Effective Structure Height $z' := 0.6 \cdot h$ $z' = 9.6\text{ ft}$
 Terrain Exposure Constant B
 (ASCE 6.5.6.3 Page 25) $z_{\min} := 15\text{-ft}$
 Equivalent structure height
 (ASCE Page 78) $z := \text{if}(z' > z_{\min}, z', z_{\min})$ $z = 15.0\text{ ft}$

Horizontal building dimension measured parallel to wind direction $L_{\text{ww}} := 21\text{-ft}$

Surface Roughness C, Building Exposure Category C.

Basic Wind Speed (Forked River, NJ (V mph)) $V_{\text{ww}} := 110$
 (ASCE page 33)

Directionality Factor $K_d := 0.85$
 (ASCE Page 80)

Building Occupancy Category $\text{Cat} := 2$
 (ASCE Page 3)

Importance Factor $I := 1.00$
 Table 6.1 (ASCE page 77)

Based on Building Occupancy Category II (ASCE page 3)

Velocity Pressure Exposure Coefficient (K_z)

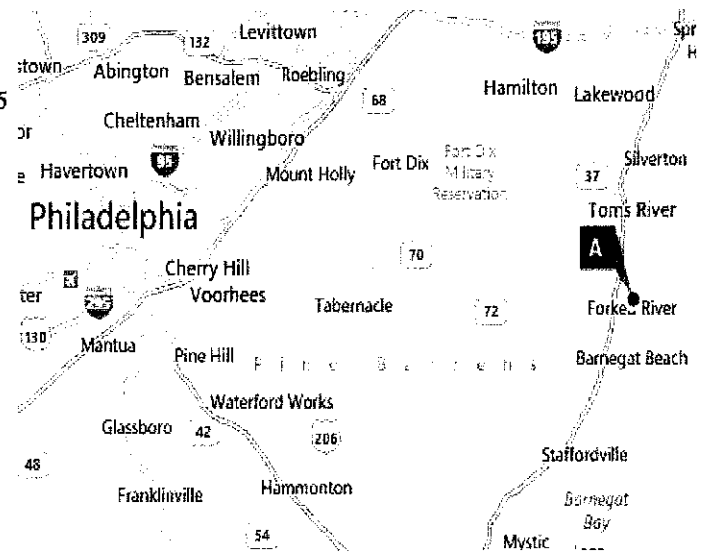
ASCE page 79, And Exposure Category C
 (Exposure category ASCE page 25)
 Construction Type Case 1

Height above ground $z_{\text{ww}} := h$ $z = 16.0\text{ ft}$

$$0.85 + \frac{0.90 - 0.85}{20\text{-ft} - 15\text{-ft}} \cdot (z - 15\text{-ft}) = 0.860$$

Interpolation of Table 6-3 for K_z

$K_z := 0.86$



Topographic Effects K_{zt}

ASCE Figure 6-4, page 45

Height of hill or escarpment relative to the upwind terrain $H_{\text{ww}} := 35\text{-ft}$

Distance Upwind/Downwind to crest to building site $x := 200\text{-ft}$

Distance upwind of crest where the difference in elevation is half the height of hill or escarpment $L_h := 150\text{-ft}$

$$\lambda_{K1} := \frac{H}{L_h}$$

$$\lambda_{K1} = 0.23$$

$$0.17 + \frac{0.21 - 0.17}{0.25 - 0.20} \cdot (\lambda_{K1} - 0.20) = 0.197$$

$$K_1 := 0.197$$

Interpolation of Figure 6-4 for K_1

$$\lambda_{K2} := \frac{x}{L_h}$$

$$\lambda_{K2} = 1.33$$

$$0.75 + \frac{0.63 - 0.75}{1.50 - 1.00} \cdot (\lambda_{K2} - 1.00) = 0.670$$

$$K_2 := 0.670$$

Interpolation of Figure 6-4 for K_2

$$\lambda_{K3} := \frac{z}{L_h}$$

$$\lambda_{K3} = 0.11$$

$$0.74 + \frac{0.55 - 0.74}{0.20 - 0.10} \cdot (\lambda_{K3} - 0.10) = 0.727$$

$$K_3 := 0.708$$

Interpolation of Figure 6-4 for K_3

$$K_{zt} := (1 + K_1 \cdot K_2 \cdot K_3)^2$$

$$K_{zt} = 1.2$$

Velocity Pressure

(ASCE page 27, eq. 6-15)

$$q_z := 0.00256 \cdot \text{psf} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$$

$$q_z = 27.1 \text{ psf}$$

Gust Effect Factor

(ASCE page 26)

FLEXIBLE STRUCTURES

Peak factor for background response

$$g_Q := 3.4$$

Peak factor for wind response

$$g_v := 3.4$$

Building Natural Frequency

$$n_1 := 1000 \text{ Hz}$$

Peak factor for resonant response

$$g_R := \sqrt{2 \cdot \ln(3600 \cdot \text{sec} \cdot n_1)} + \frac{0.577}{\sqrt{2 \cdot \ln(3600 \cdot \text{sec} \cdot n_1)}} \quad g_R = 5.6$$

Mean hourly wind speed factor

$$b' := 0.65 \quad (\text{ASCE page 78, table 6-2, Exposure C})$$

Mean hourly wind speed power law exponent

$$\alpha' := \frac{1}{6.5} \quad (\text{ASCE page 78), Exposure C}$$

Mean hourly wind speed

(eq. 6-14)

$$V_z := b' \cdot \left(\frac{z'}{33 \cdot \text{ft}} \right)^{\alpha'} \cdot V \cdot \text{mph} \cdot \frac{88}{60} \quad V_z = 127.2 \frac{\text{ft}}{\text{sec}}$$

Resonant response factor part R_h

$$\eta := 4.6 \cdot n_1 \cdot \frac{z'}{V_z}$$

$$\eta = 347.2$$

$$R_h := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta})$$

$$R_h = 0.003$$

Resonant response factor part R_B

$$\eta := 4.6 \cdot n_1 \cdot \frac{B}{V_z}$$

$$\eta = 759.5$$

$$R_B := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta})$$

$$R_B = 0.001$$

Resonant response factor part R_L

$$\eta := 15.4 \cdot n_1 \cdot \frac{L}{V_z}$$

$$\eta = 2542.6$$

$$R_L := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta})$$

$$R_L = 0.000$$

Integral length scale factor

$$L' := 500 \cdot \text{ft} \quad (\text{ASCE page 78, table 6-2, Exposure C})$$

Integral length scale power law exponent

$$\xi := \frac{1}{5.0} \quad (\text{ASCE page 78, Exposure C})$$

Turbulence intensity factor	$c := 0.2$	(ASCE page 78, Exposure C)	
Intensity of turbulence		$I_z := c \cdot \left(\frac{33\text{-ft}}{z}\right)^{\frac{1}{6}}$	$I_z = 0.2$
The integral length scale of turbulence at the equivalent height		$L_z := L' \cdot \left(\frac{z'}{33\text{-ft}}\right)^{\epsilon}$	$L_z = 390.6\text{ ft}$
Background Response		$Q := \sqrt{\frac{1}{1 + 0.63 \cdot \left(\frac{B + z'}{L_z}\right)^{0.63}}}$	$Q = 0.942$
Reduced frequency		$N_1 := \frac{n_1 \cdot L_z}{V_z}$	$N_1 = 3070.8$
Resonant response factor part R_n		$R_n := \frac{7.47 \cdot N_1}{(1 + 10.3 \cdot N_1)^{\frac{5}{3}}}$	$R_n = 0.0007$
The Resonant Response Factor	$\beta := 0.01$	$R_{\beta} := \sqrt{\frac{1}{\beta} \cdot R_n \cdot R_h \cdot R_B \cdot (0.53 + 0.47 \cdot R_L)}$	$R = 0.000$
Gust Effect Factor		$G_f := 0.925 \cdot \frac{1 + 1.7 \cdot I_z \cdot \sqrt{g_Q^2 \cdot Q^2 + g_R^2 \cdot R^2}}{1 + 1.7 \cdot g_V \cdot I_z}$	$G_f = 0.89$
External Pressure Coefficients, Windward Wall (ASCE, Figure 6.6, page 48 & 49)		$\frac{L}{B} = 1.0$	$C_p := 0.8$
Internal Pressure Coefficients for Buildings (ASCE, Figure 6.5, page 47)		$GC_{pi} := -0.00$	
Lateral Wind Pressure on windward wall (eq. 6-17, page 28)		$p' := q_z \cdot (G_f \cdot C_p - GC_{pi})$	$p' = 19.4\text{ psf}$
External Pressure Coefficients, Leeward wall (ASCE, Figure 6.6, page 49)		$\frac{L}{B} = 1.0$	$C_{pe} := -0.5$
Lateral Wind Pressure on leeward wall (eq. 6-17, page 28)		$p := q_z \cdot (G_f \cdot C_p - GC_{pi})$	$p = -12.1\text{ psf}$

3.2 Snow Load Contribution.

Exposure Factor ASCE page 92, Fully Exposed	Surface Roughness "C" Exposure Category "C" (ASCE page 25)	$C_e := 0.9$
Thermal Factor ASCE page 93	"Unheated structure"	$C_t := 1.2$
Importance Factor ASCE page 93	"Occupancy Category II" (ASCE page 3)	$I_w := 1.0$
Ground Snow Load	"Forked River, NJ " (ASCE page 85)	$p_g := 20\text{-psf}$
Flat Roof Snow load (eq. 7-1 page 81)	$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_w \cdot p_g$	$p_f = 15.1\text{ psf}$
Roof slope, run "S" for a rise of 12	$S_w := 6.93$	
Slope width	$W_w := 10\text{-ft}$	
Sloped Roof Factor ASCE page 81	Roof Slope $\theta := \frac{360}{2 \cdot \pi} \cdot \text{atan}\left(\frac{S}{12}\right)$	$\theta = 30.0$
Warm Roof Slopes ($C_t < 1.0$) ASCE page 86	$C_{s_{warm}} := \text{if}\left(\theta \leq 30, 1, 1 - \frac{\theta - 30}{70 - 30}\right)$	$C_{s_{warm}} = 1.000$
Cold Roof Slopes ($C_t = 1.1$) ASCE page 86	$C_{s_{cold1}} := \text{if}\left(\theta \leq 37.5, 1, 1 - \frac{\theta - 37.5}{70 - 37.5}\right)$	$C_{s_{cold1}} = 1.000$
Cold Roof Slopes ($C_t = 1.2$) ASCE page 86	$C_{s_{cold2}} := \text{if}\left(\theta \leq 45, 1, 1 - \frac{\theta - 45}{70 - 45}\right)$	$C_{s_{cold2}} = 1.000$
<u>Sloped Roof Factor</u>	$C_s := \text{if}(C_t \leq 1, C_{s_{warm}}, \text{if}(C_t = 1.1, C_{s_{cold1}}, C_{s_{cold2}}))$	$C_s = 1.000$
Dry Sloped Roof Snow Load	$p'_s := C_s \cdot p_f$ (eq. 7-2)	$p'_s = 15.1\text{ psf}$
Rain on Snow Surcharge ASCE page 83	$p_r := \text{if}\left(p'_s \geq 20\text{-psf} \wedge \theta \leq \frac{360}{2 \cdot \pi} \cdot \text{atan}\left(\frac{W}{50\text{-ft}}\right), 5\text{-psf}, 0\text{-psf}\right)$ if $p'_s \geq 20\text{psf}$ and $\text{roofslope} < W, 50$ use 5psf, if not use 0psf	$p_r = 0.0\text{ psf}$
Wet, Sloped Roof, Snow Load	$p_s := p'_s + p_r$	$p_s = 15.1\text{ psf}$

3.3 Dead Load Contribution.

Roof Dead Load $R_{DL} := 12.5\text{-psf}$

Wall Dead Load $W_{DL} := 5.5\text{-psf}$

3.4 Minimum Roof Live Load.

Tributary area $A_t := 59\text{-ft}^2$

Live Load reduction Factor (R_1) $R_1 := \text{if}(A_t \leq 200\text{-ft}^2, 1, \text{if}(200\text{-ft}^2 < A_t < 600\text{-ft}^2, 1.2 - 0.001 \cdot A_t, 0.6))$ $R_1 = 1.000$
ASCE, page 10

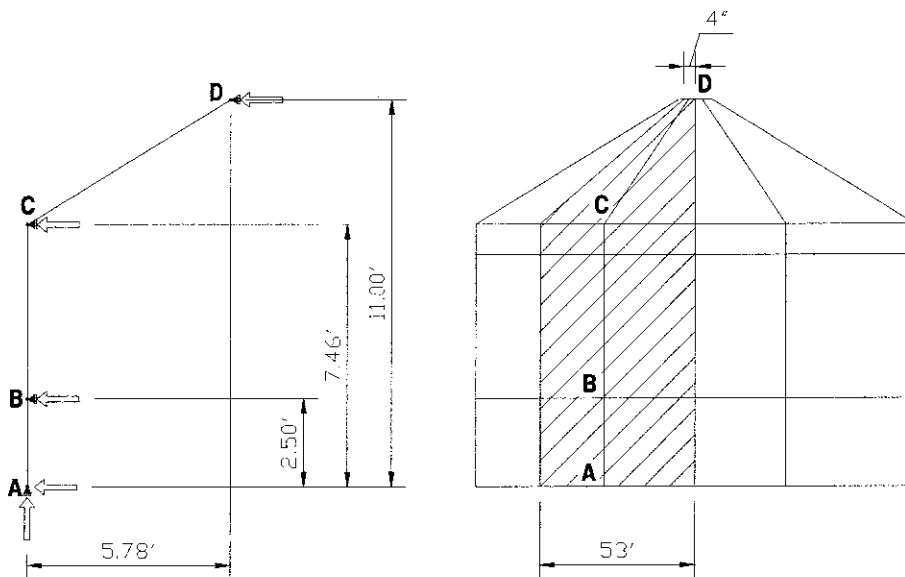
Roof slope, inches of rise per foot $F_{\frac{M}{M}} := 7$

Live Load reduction Factor (R_2) $R_2 := \text{if}(F \leq 4, 1, \text{if}(4 < F < 12, 1.2 - 0.05 \cdot F, 0.6))$ $R_2 = 0.850$
ASCE, page 11

Minimum Roof Live Loads: $R_{LL} := 20\text{-psf} \cdot R_1 \cdot R_2$ $R_{LL} = 17.0\text{ psf}$
ASCE, page 10

Since Roof Live load is less than Roof Snow Load, Use only Roof Snowload.

4.1 Determine Load Distribution on 2D Structure.



Allowable Stress Design Load Combinations
 ASCE, page 6

$D + 0.75 W + 0.75 S \text{ or } L$

Windward Wind Loads:

Structure width, center line to corner	$B_{ww} := 53\text{-in}$	
	$Wind' := 0.75 \cdot p'$	$Wind' = 14.5 \text{ psf}$
Segment AB	$Wind_{AB} := 0.6 \cdot Wind' \cdot B$	$Wind_{AB} = 38.5 \text{ plf}$
Segment BC	$Wind_{BC} := Wind' \cdot 3.5\text{-in}$	$Wind_{BC} = 4.2 \text{ plf}$
Segment CD has a triangular load distribution	$Wind_{C_{CD}} := Wind' \cdot B$	$Wind_{C_{CD}} = 64.2 \text{ plf}$
	$Wind_{D_{CD}} := Wind' \cdot 4\text{-in}$	$Wind_{D_{CD}} = 4.8 \text{ plf}$

Leeward Wind Loads:

Structure width, center line to corner	$Wind := 0.75 \cdot p$	$Wind = -9.1 \text{ psf}$
Segment AB	$Wind_{AB} := 0.6 \cdot Wind \cdot B$	$Wind_{AB} = -24.1 \text{ plf}$
Segment BC	$Wind_{BC} := Wind \cdot 3.5\text{-in}$	$Wind_{BC} = -2.6 \text{ plf}$
Segment CD has a triangular load distribution	$Wind_{C_{CD}} := Wind \cdot B$	$Wind_{C_{CD}} = -40.1 \text{ plf}$
	$Wind_{D_{CD}} := Wind \cdot 4\text{-in}$	$Wind_{D_{CD}} = -3.0 \text{ plf}$

Vertical Dead Loads:

Segment AB	$DL_{AB} := R_{DL} \cdot B$	$DL_{AB} = 55.2 \text{ plf}$
Segment CD has a triangular load distribution	$DL_{_CCD} := R_{DL} \cdot B$	$DL_{_CCD} = 55.2 \text{ plf}$
	$DL_{_DCD} := R_{DL} \cdot 4 \cdot \text{in}$	$DL_{_DCD} = 4.2 \text{ plf}$

Vertical Snow or Live:

Segment CD has a triangular load distribution	$Live_{CD} := 0.75 \cdot R_{LL} \cdot B$	$Live_{CD} = 56.3 \text{ plf}$
	$Live_{CD} := 0.75 \cdot R_{LL} \cdot 4 \cdot \text{in}$	$Live_{CD} = 4.3 \text{ plf}$

Notes:

1. Due to the small size of roof overhangs they will be assigned the same Live Load as the widest portion of the roof prior to the overhang.

4.2 Determine Load Distribution on 3D Structure.

Windward direction

Wind Load onto Hand Rail	$W_{AB} := 0.6 \cdot p'$	$W_{AB} = 11.6 \text{ psf}$
Wind Load onto Roof Fret	$W_{BC} := 0.6 \cdot p' \cdot 7 \cdot \text{in}$	$W_{BC} = 6.8 \text{ plf}$
Wind Load onto Roof	$W_{CD} := p'$	$W_{CD} = 19.4 \text{ psf}$
Wind Load on Post	$W_C := p' \cdot 3.5 \cdot \text{in}$	$W_C = 5.7 \text{ plf}$

Leeward Direction

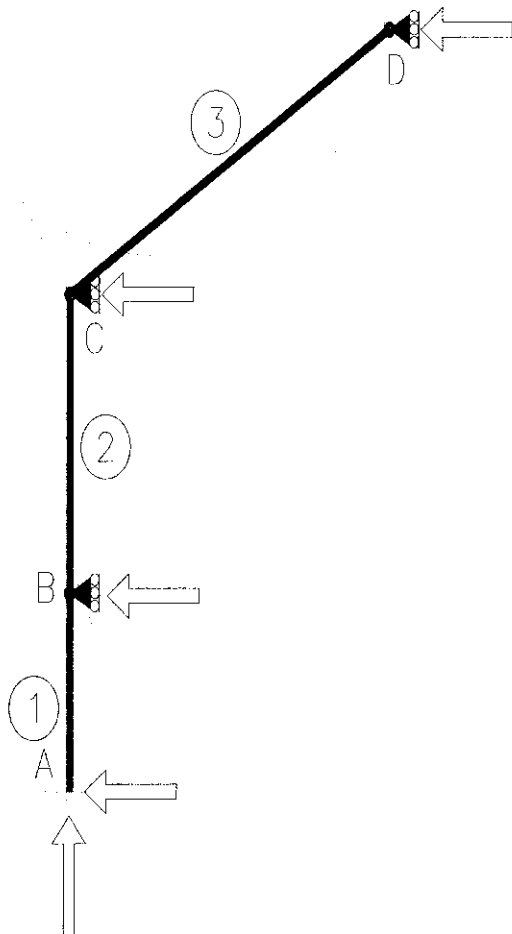
Wind Load onto Hand Rail	$W_{AB} := 0.6 \cdot p$	$W_{AB} = -7.3 \text{ psf}$
Wind Load onto Roof Fret	$W_{BC} := 0.6 \cdot p \cdot 7 \cdot \text{in}$	$W_{BC} = -4.2 \text{ plf}$
Wind Load onto Roof	$W_{CD} := p$	$W_{CD} = -12.1 \text{ psf}$
Wind Load on Post	$W_C := p \cdot 3.5 \cdot \text{in}$	$W_C = -3.5 \text{ plf}$

Snow Load onto Roof	$S_{CD} := R_{LL}$	$S_{CD} = 17.0 \text{ psf}$
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5.1 Analysis Summary.

The structure has been analyzed using a 3 dimensional analysis provided by RISA 3D. The drawing shown here is a representation of one corner of the 15' Gazebo being analyzed. The structural integrity of the Gazebo is determined by its corners which carry the loads to the ground and distribute them to all adjacent corners. The corner frame is wood construction to form an octagonal structure. The frame shown is representative of all the corners. The structure is withstanding primarily wind loads and snow loads. The magnitude of the wind forces is based on an 110 mph wind speed and the snow load on a ground snow load of 20psf. Detailed calculations are provided above. The structure is anchored by the columns being bolted to the 8" diameter concrete piers. The adjoining portions of the structure support the corners and their support is represented by a triangle and an arrow.

Lateral stability is provided by moment connections at the eaves. The moment connection connect the roof rafters and the post into a continuous member and due to the structure symmetry forms a stability triangle.



The supports acting on the corner is the support provided by adjacent building elements like hand rails, fret and fascia boards. Below is a summary of forces that these elements must be able to resist if they are to act as support for the corners.

Horizon. reactions from RISA 3D	$R_{xB} := 15\text{-lbf}$
	$R_{xC} := 32\text{-lbf}$
	$R_{xD} := 321\text{-lbf}$
Vertical Reaction, RISA 3D	$R_{y1} := 498\text{-lbf}$
Moment in Member 1, RISA 3D	$M_1 := 76\text{-ft}\cdot\text{lbf}$
Moment in Member 2, RISA 3D	$M_2 := 180\text{-ft}\cdot\text{lbf}$
Moment in Member 3, RISA 3D	$M_3 := 442\text{-ft}\cdot\text{lbf}$

Find the highest Combination of Bending stress and Shear stress. Review of structure shows highest moments in the Roof joists.

Max combined bending stress, RISA 3D	$\sigma_1 := 927\text{-psi}$
Corresponding Shear Stress	$\sigma_v := 22\text{-psi}$
Allowable Shear Stress	$F'_v = 75.0\text{ psi}$
Allowable Bending Stress	$F_{bw} = 1000.0\text{ psi}$
Safety Against Bending Failure	$\frac{F_{bw}}{\sigma_1} = 1.1 > 1.0 \text{ is OK}$
Safety Against Shear Failure	$\frac{F'_v}{\sigma_v} = 3.4 > 1.0 \text{ is OK}$

6.0 Component Analysis.

6.1 Analyze Hand Rail

Min. Live Load on Hand rails

$$i := 1..2$$

$$W_{hr} := 50 \cdot \text{plf}$$

Load decomposition angle with
 Wind perpendicular to the corner

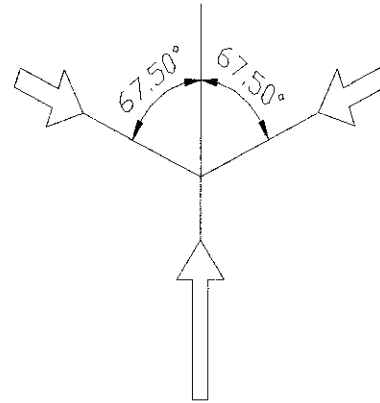
$$\alpha_1 := 67.5 \cdot \text{deg}$$

Load decomposition angle with
 Wind perpendicular to one surface

$$\alpha_2 := 45.0 \cdot \text{deg}$$

Axial force
$$P_{2_i} := \frac{R_{xB}}{\frac{2}{i} \cdot \cos(\alpha_i)}$$

$$P_2 = \begin{pmatrix} 19.6 \\ 21.2 \end{pmatrix} \text{ lbf}$$



X-sect. Area $b := 3.5 \cdot \text{in}$

$$h_m := 1.5 \cdot \text{in} \quad A_m := b \cdot h$$

$$A = 5.3 \text{ in}^2$$

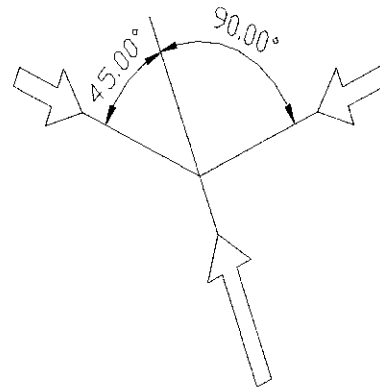
Bending Moment due to wind
 Balusters will let 30% of the wind through

$$L_w := 48 \cdot \text{in} \quad W_w := p \cdot 12 \cdot \text{in} \cdot 70\%$$

$$W = -8.5 \text{ plf}$$

Moment
$$M_{2_w} := \frac{(W_{hr} + W) \cdot L^2}{8}$$

$$M_2 = 83.0 \text{ ft} \cdot \text{lbf}$$



Section Modulus
$$S_2 := \frac{b \cdot h^2}{6}$$

$$S_2 = 1.3 \text{ in}^3$$

Threaded Rod Diameter

$$d_r := \frac{1}{2} \cdot \text{in}$$

Use 1/2" Threaded Rod in all wall Panels

Rod Area
$$A_r := \pi \cdot \frac{d_r^2}{4}$$

$$A_r = 0.2 \text{ in}^2$$

Rod Section Mod
$$S_r := \pi \cdot \frac{d_r^3}{32}$$

$$S_r = 0.012 \text{ in}^3$$

Steel Modulus of Elasticity

$$E_w := 29000 \cdot \text{ksi}$$

Steel to Wood strength Ratio

$$n := \frac{E}{E'} \quad n = 29.0$$

Check combined Compression and Bending

Max. stress
$$f_{\max_1} := \frac{P_{2_i}}{A + n \cdot A_r} + \frac{M_2}{S_2 + n \cdot S_r}$$

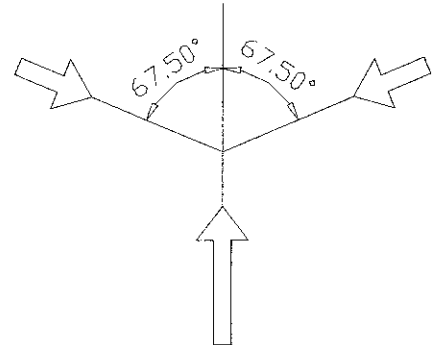
$$f_{\max} = \begin{pmatrix} 599.1 \\ 599.2 \end{pmatrix} \text{ psi}$$

Safety Factor
$$\frac{F_{bw}}{f_{\max}} = \begin{pmatrix} 1.669 \\ 1.669 \end{pmatrix} > 1.0 \text{ so OK}$$

6.2 Analyze Roof Fret

Load decomposition angle with
 Wind perpendicular to the corner

$$\alpha_1 := 67.5 \cdot \text{deg}$$



Load decomposition angle with
 Wind perpendicular to one surface

$$\alpha_2 := 45.0 \cdot \text{deg}$$

Axial force $i := 1..2$ $P_{3_i} := \frac{R_{x_C}}{\frac{2}{i} \cdot \cos(\alpha_i)}$

$$P_3 = \begin{pmatrix} 41.8 \\ 45.3 \end{pmatrix} \text{ lbf}$$

Bending Moment due to Vertical loads

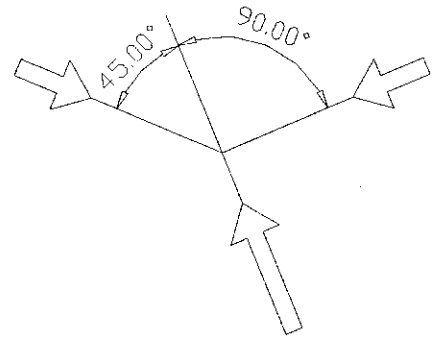
$$L_{\overline{W}} := 48 \cdot \text{in}$$

Dead Load $W_{dl} := \frac{R_{DL} \cdot B \cdot (0.71 \cdot 3 + 0.3)}{4}$

$$W_{dl} = 33.5 \text{ plf}$$

Wind Load $W_{wl} := \frac{p' \cdot \cos(30 \cdot \text{deg})^2 \cdot B \cdot (0.71 \cdot 3 + 0.3)}{4}$

$$W_{wl} = 39.0 \text{ plf}$$



Combined Vertical Load $\overline{W} := W_{dl} + W_{wl}$ $W = 72.5 \text{ plf}$

Moment $M_{3_{\overline{W}}} := \frac{W \cdot L^2}{8}$ $M_3 = 145.1 \text{ ft} \cdot \text{lbf}$

Number of Components $N_{\overline{W}} := 2$ Counter $n := 1..N$

Horizon. dim. of Bot $x_m := 3.5 \cdot \text{in}$ Vertical dim of Shape 1 $y_1 := 1.5 \cdot \text{in}$ Distance to c.g. $d_1 := \frac{y_1}{2}$

Horiz. dim. of Shape 2 $x_2 := 3.5 \cdot \text{in}$ Vertical dim of Shape 2 $y_2 := 1.5 \cdot \text{in}$ Distance to c.g. $d_2 := 12.25 \cdot \text{in} + \frac{y_2}{2}$

Overall Height of this section $H_{\overline{W}} := 13.75 \cdot \text{in}$

Determine Location of Neutral Axis of Lower Roof Fret

Component Areas $At'_n := x_n \cdot y_n$

Gross Area $At := \sum_n At'_n$ $At = 10.5 \text{ in}^2$

Distance from base to N.A. $D_{na} := \frac{\sum_n [At'_n \cdot (d_n)]}{\sum_n At'_n}$ $D_{na} = 6.9 \text{ in}$

Distance from NA to c.g. of components $y' := D_{na} - d$

Moment of Inertia $It := \sum_n \left[x_n \cdot y_n \cdot (y'_n)^2 + \frac{x_n \cdot (y_n)^3}{12} \right]$ $It = 395.9 \text{ in}^4$

Section Modulus Top $Sf_1 := \frac{It}{H - D_{na}}$ Bottom $Sf_2 := \frac{It}{D_{na}}$ $Sf = \left(\frac{57.6}{57.6} \right) \text{ in}^3$

Check combined Compression and Bending

Max. stress $f_{max_i} := \frac{P_{3_i}}{At} + \frac{M_3}{Sf_1}$ $f_{max} = \left(\frac{34.2}{34.5} \right) \text{ psi}$ Safety Factor $\frac{Fb_w}{f_{max}} = \left(\frac{29.228}{28.951} \right) > 1.0 \text{ so OK}$

7.1 Safety Against Sliding when built on Concrete Slab.

Effective Projected Area of Garden House

Area of Hand rails	$Ag_1 := 181\text{-in}\cdot 30\text{-in}\cdot 70\text{-}\%$	$Ag_1 = 26.4\text{ ft}^2$	DST to center of g.	$d_1 := \frac{30\text{-in}}{2}$
Area of Low Fret	$Ag_2 := 160\text{-in}\cdot 14\text{-in}\cdot 70\text{-}\%$	$Ag_2 = 10.9\text{ ft}^2$	Dist to center of g.	$d_2 := 87\text{-in}$
Area of Roof	$Ag_3 := \frac{(181\text{-in} + 98\text{-in})}{2}\cdot 65\text{-in}$	$Ag_3 = 63.0\text{ ft}^2$	Dist to center of g.	$d_3 := 105\text{-in}$
Area of Cupola	$Ag_4 := 28\text{-in}\cdot 18\text{-in}$	$Ag_4 = 3.5\text{ ft}^2$	Dist to center of g.	$d_4 := 175\text{-in}$
Total projected Area	$Ag_{tot} := \sum Ag$			$Ag_{tot} = 103.8\text{ ft}^2$

Lateral Wind Force $V_{tot} := (p' + p)\cdot Ag_{tot}$ $V_{tot} = 753.9\text{ lbf}$

Assume some common soil parameters

Angle of Internal Friction (estimated) $\theta_{ww} := 15\text{-deg}$

Coefficient of Cohesion (estimated) $c_{ww} := 0.2\text{-ksf}$

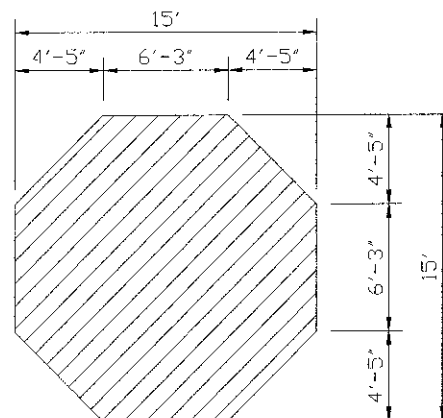
Concrete Slab parameters

Slab thickness $t_{slab} := 4\text{-in}$

Area of 8" thick Concrete Slab

$A_{slab} := (180\text{-in})^2 - 2\cdot(53\text{-in})^2$ $A_{slab} = 186.0\text{ ft}^2$

Load combination Wind + 0.6 Dead Load



Weight of Concrete Slab $W_{conc} := t_{slab}\cdot A_{slab}\cdot \rho_{con}$ $W_{conc} = 9299.3\text{ lbf}$

Determine the Friction-cohesion resistance $Fr := W_{conc}\cdot \tan(\theta) + \frac{2}{3}\cdot c\cdot A_{slab}$ $Fr = 27289.9\text{ lbf}$

Safety factor $\frac{Fr}{V_{tot}} = 36.2 > 1.5$ is **OK**

7.2 Safety Against Overturning when built on Concrete Slab

Overturning Moment due to Wind $M_o := (p' + p)\cdot (Ag\cdot d)$ $M_o = 5188\text{ ft}\cdot\text{lbf}$

Stabilizing Moment due to concrete slab

$B_{ww} := 180\text{-in}$ $M_s := W_{conc}\cdot \frac{B}{2}$ $M_s = 69745\text{ ft}\cdot\text{lbf}$

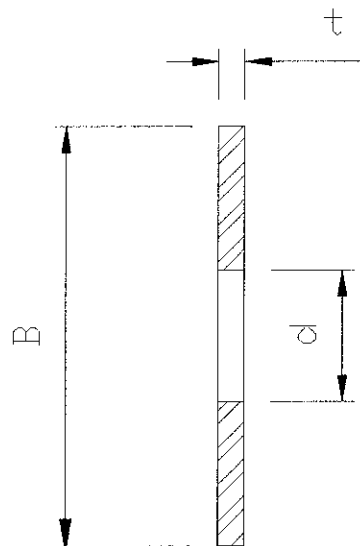
Safety factor $\frac{M_s}{M_o} = 13.4 > 2.0$ is **OK**

8. Column Base Strap Design.

$$B_w := 2 \cdot \text{in}$$

$$d := \frac{5}{8} \cdot \text{in}$$

$$t := \frac{1}{8} \cdot \text{in}$$



Total Vertical Load $V_d := 1450 \cdot \text{lbf}$

Design Moment $M_d := 948 \cdot \text{in} \cdot \text{lbf}$

Modulus of Inertia of Bracket at the hole $I_h := 0.12119 \cdot \text{in}^4$

Section Modulus $S_h := \frac{2 \cdot I_h}{B}$ $S_h = 0.121 \text{ in}^3$

Cross Section Area at Hole $A_h := (B - d) \cdot t$ $A_h = 0.172 \text{ in}^2$

Stress in bracket $\sigma_{bracket} := \frac{M_d}{S_h} + \frac{V_d}{A_h}$ $\sigma_1 = 16.3 \text{ ksi}$

Allowable Stress in Bracket $F_{all} := 0.6 \cdot F_{yA36}$ $F_{all} = 21.6 \text{ ksi}$

Safety Factor $\frac{F_{all}}{\sigma_1} = 1.3 > 1.0$ is OK

Number of Bolts $N_o := 4$ Counter $n := 1 \dots N_o$

Thickness of Bracket $t = 0.125 \text{ in}$

Diameter & Area per Bolt $d' := \frac{1}{2} \cdot \text{in}$ $A_t' := d' \cdot t$

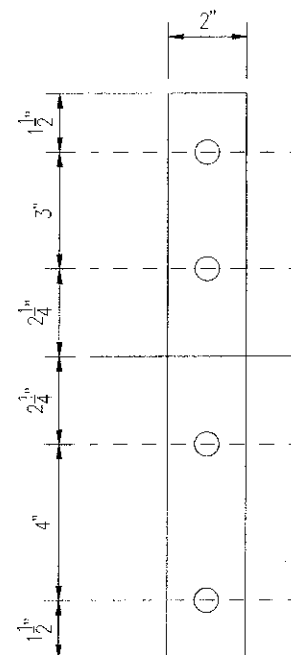
Bolt Moment of Inertia $I_n := \frac{t \cdot d'^3}{12}$ $I_1 = 0.0013 \text{ in}^4$

Vertical dist to cg of Bolt 1 $d_1 := 0 \cdot \text{in}$

Vertical dist to cg of Bolt 2 $d_2 := 4 \cdot \text{in}$

Vertical dist to cg of Bolt 3 $d_3 := 8.5 \cdot \text{in}$

Vertical dist to cg of Bolt 4 $d_4 := 11.5 \cdot \text{in}$



Distance from base to Nuteral Axis		$D_{na} := \frac{\sum [At'_n \cdot (d_n)]}{\sum At'_n}$	$D_{na} = 6.00 \text{ in}$
Dist. from NA to c.g. of blot group	$y' := D_{na} - d$		
Bolt Group Moment of Inertia	$I_t := \sum_n [At'_n \cdot (y'_n)^2 + I_n]$	$I_t = 4.786 \text{ in}^4$	
Bolt Group Section modulus		$S := \frac{I_t}{D_{na}}$	$S = 0.798 \text{ in}^3$
Maximum Bolt Shear	$V_{max} := \frac{M_d}{S} \cdot At'_1$		$V_{max} = 74.3 \text{ lbf}$
Wood thickness	$t_{wood} := 1.5 \cdot \text{in}$		
Maximum, Bearing Stress	$\sigma_{bear} := \frac{V_{max}}{d' \cdot t_{wood}}$	$\sigma_{bear} = 99.0 \text{ psi}$	
Allowable compression stress perpendicular to grain		$F_{cL} = 425.0 \text{ psi}$	
	Safety Factor	$\frac{F_{cL}}{\sigma_{bear}} = 4.3 > 1.0 \text{ is OK}$	