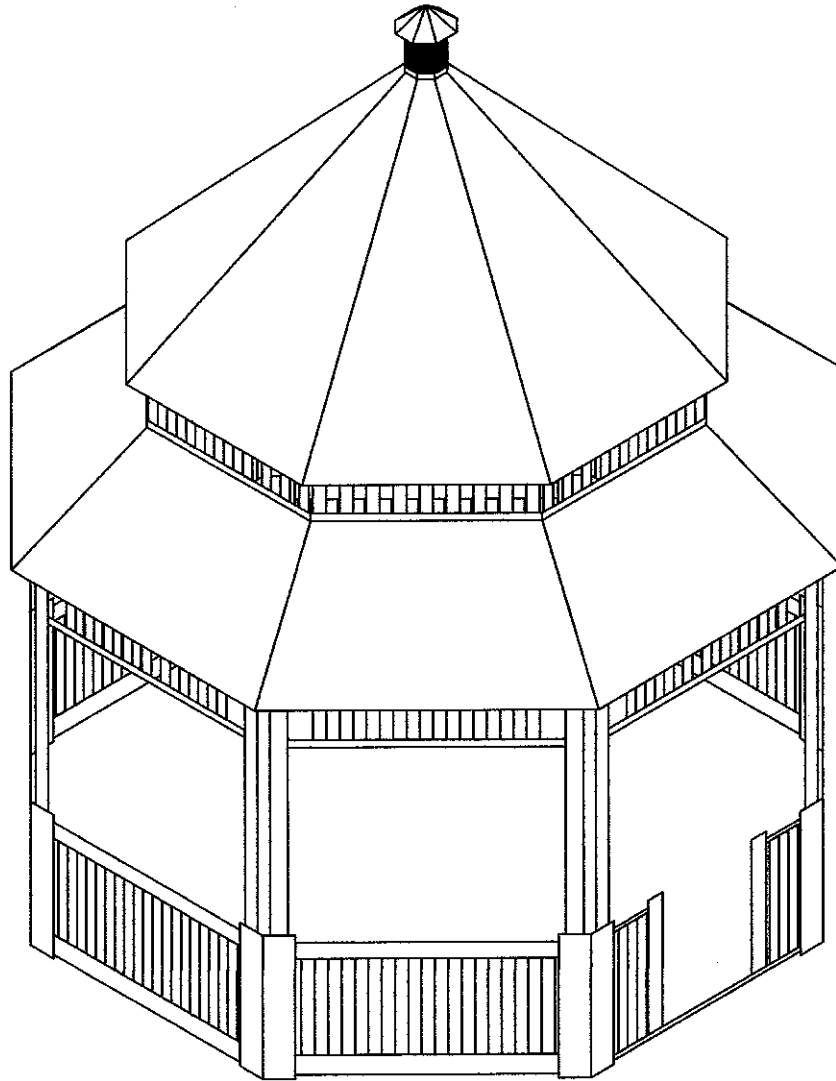


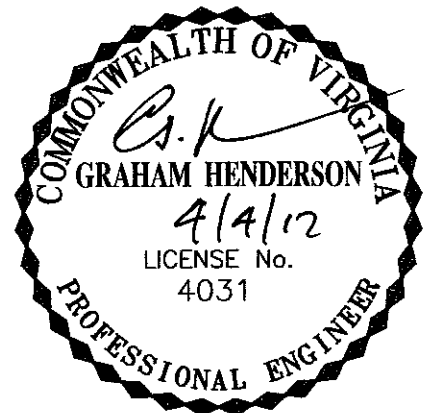
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## STRUCTURAL ANALYSIS MODEL 21' PAVILION



**LACEY SUBDIVISON  
FORKED RIVER, NJ**



<b>CLIENT</b>	<b>Toll Brothers, Inc.</b>
<b>PROJECT</b>	<b>Forked River, New Jersey</b>
<b>JOB No:</b>	<b>12209</b>
<b>CALCULATED BY</b>	<b>Robert W. Riiber, P.E.</b>
<b>CHECKED BY</b>	<b>Graham Henderson, P.E., L.S.</b>
<b>STRUCTURE</b>	<b>Vixen Hill - 21' Pavilion</b>

## CALCULATION INDEX

1. Units.
2. Materials, Codes and Assumptions.
3. Load Determinations.
4. Determine Load Distribution on Structure.
5. Analysis Summary.
6. Component Analysis.
7. Safety Against Overturning when built on Concrete Slab.
8. Additional Overturning Stability.
9. Column Base Strap Design.



**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  $h := 25\text{-ft}$   
 Effective Structure Height  $z' := 0.6 \cdot h$   $z' = 15.0\text{ ft}$   
 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  $L_{\text{W}} := 21\text{-ft}$

Surface Roughness C, Building Exposure Category C.

Basic Wind Speed (Forked River, NJ (V mph))  $V_{\text{W}} := 110$   
 (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 ( $K_z$ )

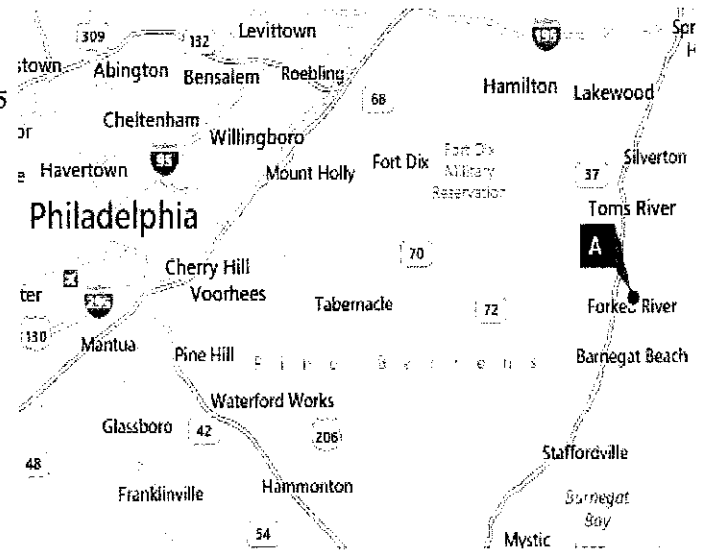
ASCE page 79, And Exposure Category C  
 (Exposure category ASCE page 25)  
 Construction Type Case 1

Height above ground  $z := h$   $z = 25.0\text{ ft}$

$$0.85 + \frac{0.90 - 0.85}{20\text{-ft} - 15\text{-ft}} \cdot (z - 15\text{-ft}) = 0.950$$

Interpolation of Table 6-3 for  $K_z$

$K_z := 0.94$



Topographic Effects  $K_{zt}$

ASCE Figure 6-4, page 45

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

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  $L_h := 150\text{-ft}$

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

$$\lambda_{K1} = 0.23$$

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

$$K_1 := 0.197$$

Interpolation of Figure 6-4 for  $K_1$

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

$$\lambda_{K2} = 1.33$$

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

$$K_2 := 0.670$$

Interpolation of Figure 6-4 for  $K_2$

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

$$\lambda_{K3} = 0.17$$

$$0.74 + \frac{0.55 - 0.74}{0.20 - 0.10} (\lambda_{K3} - 0.10) = 0.613$$

$$K_3 := 0.708$$

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)

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

$$q_z = 29.6 \text{ psf}$$

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)

$$V_z := b' \cdot \left( \frac{z'}{33 \cdot \text{ft}} \right)^{\alpha'} \cdot V \cdot \text{mph} \cdot \frac{88}{60} \quad V_z = 136.2 \frac{\text{ft}}{\text{sec}}$$

Resonant response factor part  $R_h$

$$\eta := 4.6 \cdot n_1 \cdot \frac{z'}{V_z} \quad \eta = 506.5$$

$$R_h := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta}) \quad R_h = 0.002$$

Resonant response factor part  $R_B$

$$\eta_w := 4.6 \cdot n_1 \cdot \frac{B}{V_z} \quad \eta = 709.1$$

$$R_B := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta}) \quad R_B = 0.001$$

Resonant response factor part  $R_L$

$$\eta_w := 15.4 \cdot n_1 \cdot \frac{L}{V_z} \quad \eta = 2373.8$$

$$R_L := \frac{1}{\eta} - \frac{1}{2 \cdot \eta^2} \cdot (1 - e^{-2 \cdot \eta}) \quad R_L = 0.000$$

Integral length scale factor

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

Integral length scale power law exponent

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

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	$L_z := L' \cdot \left(\frac{z'}{33\text{-ft}}\right)^{\epsilon}$		$L_z = 427.1\text{ ft}$
Background Response	$Q := \sqrt{\frac{1}{1 + 0.63 \cdot \left(\frac{B + z'}{L_z}\right)^{0.63}}}$		$Q = 0.94$
Reduced frequency	$N_1 := \frac{n_1 \cdot L_z}{V_z}$		$N_1 = 3134.7$
Resonant response factor part $R_n$	$R_n := \frac{7.47 \cdot N_1}{\frac{5}{(1 + 10.3 \cdot N_1)^3}}$		$R_n = 0.0007$
The Resonant Response Factor	$\beta := 0.01$	$R_w := \sqrt{\frac{1}{\beta} \cdot R_n \cdot R_h \cdot R_B \cdot (0.53 + 0.47 \cdot R_L)}$	$R = 0.000$
<b>Gust Effect Factor</b>	$G_f := 0.925 \cdot \frac{1 + 1.7 \cdot I_z \cdot \sqrt{gQ^2 \cdot Q^2 + gR^2 \cdot R^2}}{1 + 1.7 \cdot g_v \cdot I_z}$		$G_f = 0.89$
External Pressure Coefficients, <b>Windward Wall</b> (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)	$p' := q_z \cdot (G_f \cdot C_p - GC_{pi})$		$p' = 21.2\text{ psf}$
External Pressure Coefficients, <b>Leeward wall</b> (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)	$p := q_z \cdot (G_f \cdot C_p - GC_{pi})$		$p = -13.2\text{ psf}$

### 3.2 Snow Load Contribution.

<b>Exposure Factor</b> ASCE page 92, Fully Exposed	Surface Roughness "C" Exposure Category "C" (ASCE page 25)	$C_e := 0.9$
<b>Thermal Factor</b> ASCE page 93	"Unheated structure"	$C_t := 1.2$
<b>Importance Factor</b> ASCE page 93	"Occupancy Category II" (ASCE page 3)	$I_w := 1.0$
<b>Ground Snow Load</b>	"Forked River, NJ " (ASCE page 85)	$p_g := 20\text{-psf}$
<b>Flat Roof Snow load</b> (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	$\frac{S}{W} := 6.93$	
Slope width	$\frac{W}{W} := 10\text{-ft}$	
<b>Sloped Roof Factor</b> 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	$C_{s_{\text{warm}}} := \text{if}\left(\theta \leq 30, 1, 1 - \frac{\theta - 30}{70 - 30}\right)$	$C_{s_{\text{warm}}} = 1.000$
Cold Roof Slopes ( $C_t = 1.1$ ) ASCE page 86	$C_{s_{\text{cold1}}} := \text{if}\left(\theta \leq 37.5, 1, 1 - \frac{\theta - 37.5}{70 - 37.5}\right)$	$C_{s_{\text{cold1}}} = 1.000$
Cold Roof Slopes ( $C_t = 1.2$ ) ASCE page 86	$C_{s_{\text{cold2}}} := \text{if}\left(\theta \leq 45, 1, 1 - \frac{\theta - 45}{70 - 45}\right)$	$C_{s_{\text{cold2}}} = 1.000$
<u>Sloped Roof Factor</u>	$C_s := \text{if}(C_t \leq 1, C_{s_{\text{warm}}}, \text{if}(C_t = 1.1, C_{s_{\text{cold1}}}, C_{s_{\text{cold2}}}))$	$C_s = 1.000$
<b>Dry Sloped Roof Snow Load</b>	$p'_s := C_s \cdot p_f$ (eq. 7-2)	$p'_s = 15.1\text{ psf}$
<b>Rain on Snow Surcharge</b> 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	$p_r = 0.0\text{ psf}$
<b>Wet, Sloped Roof, Snow Load</b>	$p_s := p'_s + p_r$	$p_s = 15.1\text{ psf}$

### 3.3 Dead Load Contribution.

Roof Dead Load  $R_{DL} := 12.5\text{-psf}$   
Wall Dead Load  $W_{DL} := 1.5\text{-psf}$

### 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  $\frac{F}{\Delta X} := 7$

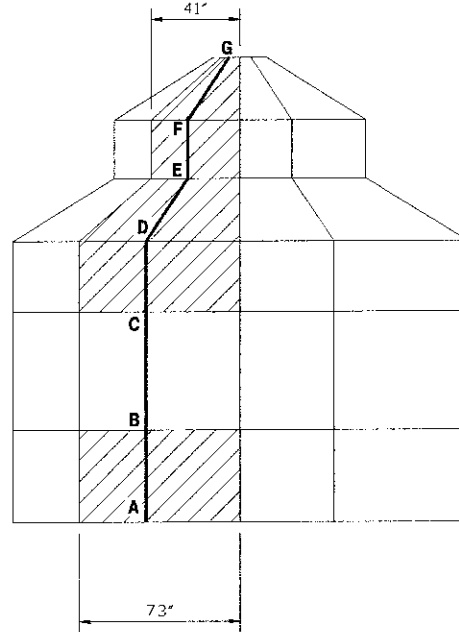
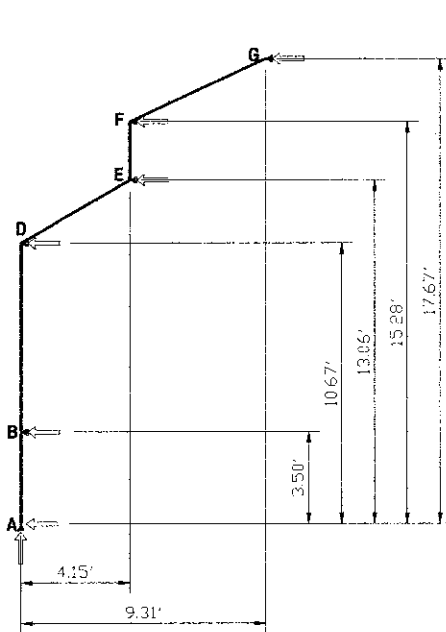
Live Load reduction Factor ( $R_2$ )  $R_2 := \text{if}(F \leq 4, 1, \text{if}(4 < F < 12, 1.2 - 0.05 \cdot F, 0.6))$   $R_2 = 0.850$   
ASCE, page 11

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 that 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:**

Influence width at Base

$$B1 := 73\text{-in}$$

Influence width at Fret

$$B2 := 41\text{-in}$$

$$\text{Wind}' := 0.75 \cdot p'$$

$$\text{Wind}' = 15.9 \text{ psf}$$

Segment AB

$$\text{Wind}_{AB} := 0.6 \cdot \text{Wind}' \cdot B1$$

$$\text{Wind}_{AB} = 58.0 \text{ plf}$$

Segment BC

$$\text{Wind}_{BC} := \text{Wind}' \cdot 3.5\text{-in}$$

$$\text{Wind}_{BC} = 4.6 \text{ plf}$$

Segment CD

$$\text{Wind}_{CD} := 0.6 \text{Wind}' \cdot B1$$

$$\text{Wind}_{CD} = 58.0 \text{ plf}$$

Segment DE has a triangular load distribution

$$\text{Wind}_{ADE} := \text{Wind}' \cdot B1$$

$$\text{Wind}_{ADE} = 96.6 \text{ plf}$$

$$\text{Wind}_{BDE} := \text{Wind}' \cdot B2$$

$$\text{Wind}_{BDE} = 54.3 \text{ plf}$$

Segment EF

$$\text{Wind}_{EF} := 0.6 \cdot \text{Wind}' \cdot B2$$

$$\text{Wind}_{EF} = 32.6 \text{ plf}$$

Segment FG has a triangular load distribution

$$\text{Wind}_{AFG} := \text{Wind}' \cdot B2$$

$$\text{Wind}_{AFG} = 54.3 \text{ plf}$$

$$\text{Wind}_{BFG} := \text{Wind}' \cdot 6\text{-in}$$

$$\text{Wind}_{BFG} = 7.9 \text{ plf}$$

**Vertical Dead Loads:**

Segment AB	$DL_{AB} := W_{DL} \cdot 3.5\text{-ft}$	$DL_{AB} = 5.3\text{ plf}$
Segment CD	$DL_{CD} := W_{DL} \cdot 14\text{-in}$	$DL_{CD} = 1.8\text{ plf}$
Segment DE has a triangular load distribution	$DL_{A_{DE}} := R_{DL} \cdot B1$	$DL_{A_{DE}} = 76.0\text{ plf}$
	$DL_{B_{DE}} := R_{DL} \cdot B2$	$DL_{B_{DE}} = 42.7\text{ plf}$
Segment EF	$DL_{EF} := W_{DL} \cdot 14\text{-in}$	$DL_{EF} = 1.8\text{ plf}$
Segment FG has a triangular load distribution	$DL_{A_{FG}} := R_{DL} \cdot B2$	$DL_{A_{FG}} = 42.7\text{ plf}$
	$DL_{B_{FG}} := R_{DL} \cdot 6\text{-in}$	$DL_{B_{FG}} = 6.3\text{ plf}$

**Vertical Snow or Live:**

Segment DE has a triangular load distribution	$Live_{A_{DE}} := 0.75 \cdot R_{LL} \cdot B1$	$Live_{A_{DE}} = 77.6\text{ plf}$
	$Live_{B_{DE}} := 0.75 \cdot R_{LL} \cdot B2$	$Live_{B_{DE}} = 43.6\text{ plf}$
Segment FG has a triangular load distribution	$Live_{A_{FG}} := 0.75 \cdot R_{LL} \cdot B2$	$Live_{A_{FG}} = 43.6\text{ plf}$
	$Live_{B_{FG}} := 0.75 \cdot R_{LL} \cdot 6\text{-in}$	$Live_{B_{FG}} = 6.4\text{ plf}$

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	$W_{AB} := 0.5\text{-p'}$	$W_{AB} = 10.6\text{ psf}$
Wind Load onto Roof Fret	$W_{BC} := 0.5\text{-p' \cdot 7-in}$	$W_{BC} = 6.2\text{ plf}$
Wind Load onto Roof	$W_{CD} := p'$	$W_{CD} = 21.2\text{ psf}$
Wind Load on Post	$W_C := p' \cdot 3.5\text{-in}$	$W_C = 6.2\text{ plf}$

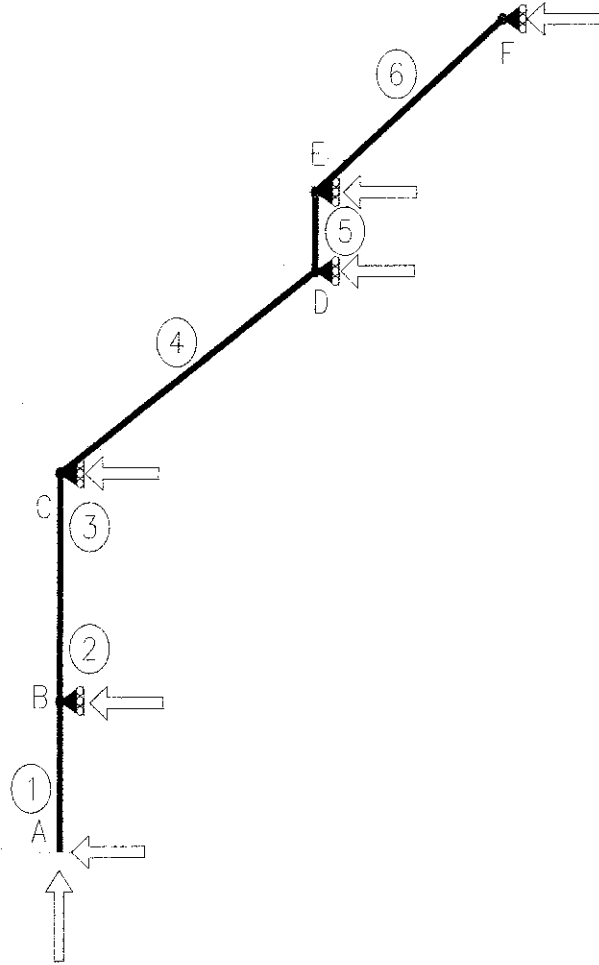
Leeward Direction

Wind Load onto Hand Rail	$\overline{W}_{AB} := 0.5\text{-p}$	$W_{AB} = -6.6\text{ psf}$
Wind Load onto Roof Fret	$\overline{W}_{BC} := 0.5\text{-p \cdot 7-in}$	$W_{BC} = -3.9\text{ plf}$
Wind Load onto Roof	$\overline{W}_{CD} := p$	$W_{CD} = -13.2\text{ psf}$
Wind Load on Post	$\overline{W}_C := p \cdot 3.5\text{-in}$	$W_C = -3.9\text{ plf}$
Snow Load onto Roof	$S_{CD} := R_{LL}$	$S_{CD} = 17.0\text{ psf}$

### 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 21' 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	$R_{xB} := 42\text{-lbf}$
	$R_{xC} := 589\text{-lbf}$
	$R_{xD} := 1061\text{-lbf}$
Vertical Reaction, (N1) RISA 3D	$R_{y1} := 2057\text{-lbf}$
Moment in Member 1, (M24) RISA 3D	$M_1 := 4045\text{-ft}\cdot\text{lbf}$
Moment in Member 2, (M23) RISA 3D	$M_2 := 1846\text{-ft}\cdot\text{lbf}$
Moment in Member 3, (M22) RISA 3D	$M_3 := 2685\text{-ft}\cdot\text{lbf}$
Moment in Member 4, (M20) RISA 3D	$M_4 := 26\text{-ft}\cdot\text{lbf}$

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

Max Bending Stress, (M63) RISA 3D	$\sigma_1 := 1004\text{-psi}$
Max. Shear Stress	$\sigma_v := 55\text{-psi}$
Allowable Shear Stress	$F'_v = 75.0\text{ psi}$
Allowable Bending Stress	$F_{bw} = 1000.0\text{ psi}$

Safety Against Bending Failure  $\frac{F_{bw}}{\sigma_1} = 1.00 \geq 1.0$  is **OK**

Safety Against Shear Failure  $\frac{F'_v}{\sigma_v} = 1.4 > 1.0$  is **OK**

## 6.0 Component Analysis.

### 6.1 Analyze Hand Rail

Min. Live Load on Hand rails

$$i := 1..2$$

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

Load decomposition angle with  
 Wind perpendicular to the corner

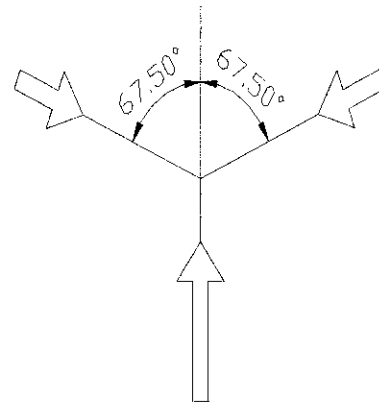
$$\alpha_1 := 67.5\text{-deg}$$

Load decomposition angle with  
 Wind perpendicular to one surface

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

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

$$P_2 = \begin{pmatrix} 54.9 \\ 59.4 \end{pmatrix} \text{ lbf}$$



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

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

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

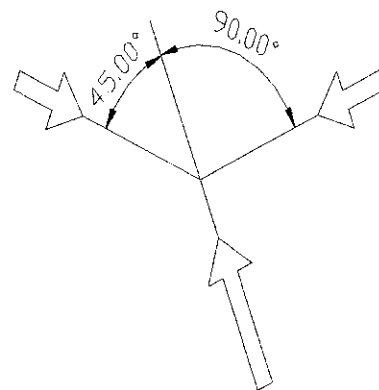
Bending Moment due to wind  
 Balusters will let 30% of the wind through

$$L_w := 48\text{-in} \quad W_w := p \cdot 12\text{-in} \cdot 50\text{-\%}$$

$$W = -6.6 \text{ plf}$$

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

$$M_2 = 86.8 \text{ ft}\cdot\text{lbf}$$



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

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

Threaded Rod Diameter

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

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

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

Steel to Wood strength Ratio

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

### Check combined Compression and Bending

Max. stress 
$$f_{\max_i} := \frac{P_{2_i}}{A + n \cdot A_r} + \frac{M_2}{S_2 + n \cdot S_r} \quad f_{\max} = \begin{pmatrix} 629.1 \\ 629.5 \end{pmatrix} \text{ psi}$$

Safety Factor 
$$\frac{F_{bw}}{f_{\max}} = \begin{pmatrix} 1.59 \\ 1.589 \end{pmatrix} > 1.0 \text{ so OK}$$

## 6.2 Analyze Roof Fret

Load decomposition angle with  
 Wind perpendicular to the corner

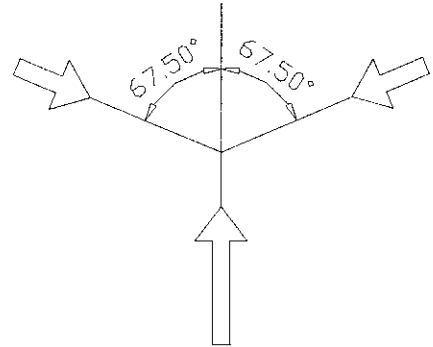
$$\alpha_1 := 67.5\text{-deg}$$

Load decomposition angle with  
 Wind perpendicular to one surface

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

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

$$P_3 = \begin{pmatrix} 769.6 \\ 833.0 \end{pmatrix} \text{ lbf}$$



Bending Moment due to Vertical loads

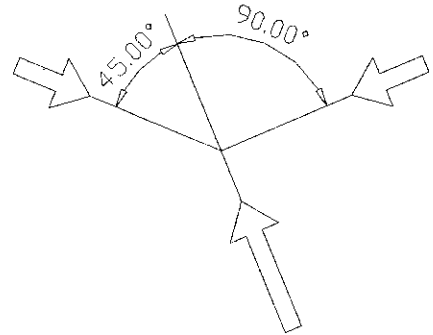
$$L_{\overline{w}} := 48\text{-in}$$

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

$$W_{dl} = 159.5 \text{ plf}$$

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

$$W_{wl} = 202.6 \text{ plf}$$



Combined Vertical Load

$$W_{\overline{w}} := W_{dl} + W_{wl}$$

$$W = 362.1 \text{ plf}$$

Moment

$$M_{3_i} := \frac{W \cdot L^2}{8}$$

$$M_3 = 724.1 \text{ ft-lbf}$$

Number of Components  $N_{\overline{w}} := 2$

Counter  $n_{\overline{w}} := 1..N$

Horizon. dim. of Bot

$$x_{M1} := 3.5\text{-in}$$

Vertical dim of Shape 1

$$y_1 := 1.5\text{-in}$$

Distance to c.g.  $d_1 := \frac{y_1}{2}$

Horiz. dim. of Shape 2

$$x_2 := 3.5\text{-in}$$

Vertical dim of Shape 2

$$y_2 := 1.5\text{-in}$$

Distance to c.g.  $d_2 := 12.25\text{-in} + \frac{y_2}{2}$

Overall Height of this section

$$H_{\overline{w}} := 13.75\text{-in}$$

**Determine Location of Neutral Axis of Lower Roof Fret**

Component Areas  $A_{n'} := x_n \cdot y_n$

Gross Area  $A_t := \sum_n A_{n'}$   $A_t = 10.5 \text{ in}^2$

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

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

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

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

**Check combined Compression and Bending**

Max. stress  $f_{max_i} := \frac{P_{3_i}}{A_t} + \frac{M_3}{Sf_1}$   $f_{max} = \left( \frac{224.2}{230.2} \right) \text{ psi}$  Safety Factor  $\frac{Fb_w}{f_{max}} = \left( \frac{4.46}{4.343} \right) > 1.0 \text{ so OK}$

## 7.1 Safety Against Sliding when built on Concrete Slab.

### Effective Projected Area of 21' Pavilion

Area of Hand rails	$Ag_1 := 237\text{-in} \cdot 42\text{-in} \cdot 50\%$	$Ag_1 = 34.6\text{ ft}^2$	DST to center of g.	$d_1 := 28\text{-in}$
Area of Low Fret	$Ag_2 := 237\text{-in} \cdot 14\text{-in} \cdot 50\%$	$Ag_2 = 11.5\text{ ft}^2$	Dist to center of g.	$d_2 := 117\text{-in}$
Area of Low Roof	$Ag_3 := \frac{(264\text{-in} + 177\text{-in})}{2} \cdot 44\text{-in}$	$Ag_3 = 67.4\text{ ft}^2$	Dist to center of g.	$d_3 := 140\text{-in}$
Area of Riser	$Ag_4 := 177\text{-in} \cdot 18\text{-in} \cdot 50\%$	$Ag_4 = 11.1\text{ ft}^2$	Dist to center of g.	$d_4 := 170\text{-in}$
Area of High Roof	$Ag_5 := \frac{(191\text{-in} + 13\text{-in})}{2} \cdot 52\text{-in}$	$Ag_5 = 36.8\text{ ft}^2$	Dist to center of g.	$d_5 := 196\text{-in}$

Total projected Area  $Ag_{tot} := \sum Ag$   $Ag_{tot} = 161.4\text{ ft}^2$

Lateral Wind Force

$V_{tot} := (p' + p) \cdot Ag_{tot}$

$V_{tot} = 1281.2\text{ lbf}$

Assume some common soil parameters

Angle of Internal Friction (estimated)

$\theta := 15\text{-deg}$

Coefficient of Cohesion (estimated)

$c := 0.2\text{-ksf}$

Concrete Slab parameters

**Slab thickness**

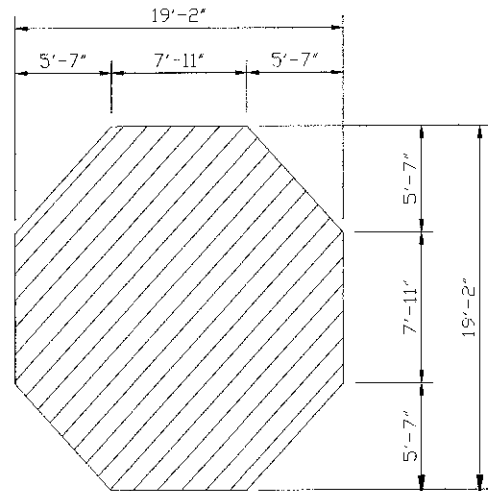
$t_{slab} := 8\text{-in}$

Area of 8" thick Air Entrained Concrete Slab with 6x6- W1.4xW1.4 Mesh

$A_{slab} := (230\text{-in})^2 - 2 \cdot (67\text{-in})^2$

$A_{slab} = 305.0\text{ ft}^2$

Load combination Wind + 0.6 Dead Load



Weight of Concrete Slab

$W_{conc} := 0.6 \cdot t_{slab} \cdot A_{slab} \cdot \rho_{con}$

$W_{conc} = 18300.8\text{ lbf}$

Determine the Friction-cohesion resistance

$Fr := W_{conc} \cdot \tan(\theta) + \frac{2}{3} \cdot c \cdot A_{slab}$

$Fr = 45572.2\text{ lbf}$

Safety factor

$\frac{Fr}{V_{tot}} = 35.6$

> 1.5 is OK

## 7.2 Safety Against Overturning when built on Concrete Slab

Overturning Moment due to Wind  $M_o := (p' + |p|) \cdot (Ag \cdot d)$   $M_o = 59778 \text{ ft} \cdot \text{lb}$

Stabilizing Moment due to concrete slab

$$B_w := 230 \cdot \text{in}$$

$$M_s := W_{\text{conc}} \cdot \frac{B}{2} \quad M_s = 175383 \text{ ft} \cdot \text{lb}$$

Safety factor  $\frac{M_s}{M_o} = 2.9 > 2.0 \text{ is OK}$

## 8. Column Base Strap Design.

$$B_w := 3.5 \cdot \text{in}$$

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

$$t := \frac{3}{16} \cdot \text{in}$$

Total Vertical Load  $V_d := 2139 \cdot \text{lb}$

Design Moment  $M_d := M_1 \quad M_d = 48.5 \text{ in} \cdot \text{kip}$

Modulus of Inertia of Bracket at the hole  $I_h := 20.66 \cdot \text{in}^4$

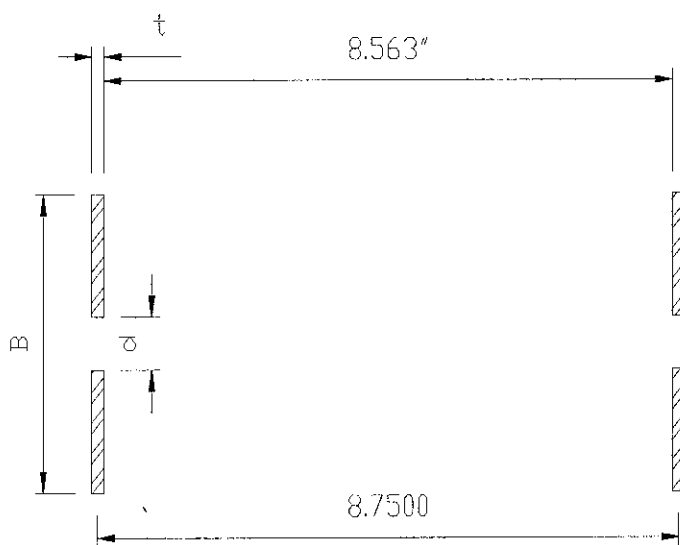
Section Modulus  $S_h := \frac{2 \cdot I_h}{5.75 \cdot \text{in}} \quad S_h = 7.186 \text{ in}^3$

Cross Section Area at Hole  $A_h := 2 \cdot (B - d) \cdot t \quad A_h = 1.078 \text{ in}^2$

Stress in bracket  $\sigma_{\text{all}} := \frac{M_d}{S_h} + \frac{V_d}{A_h} \quad \sigma_1 = 8.7 \text{ ksi}$

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

Safety Factor  $\frac{F_{\text{all}}}{\sigma_1} = 2.47 > 1.0 \text{ is OK}$





Moment imposed on weld	$M := M_d$	$M = 48.5 \text{ in}\cdot\text{kip}$	
Vertical Load on weld	$V_w := 2.07 \cdot \text{kip} + \frac{M_d}{8.75 \cdot \text{in}}$	$V = 7.6 \text{ kip}$	
Horizontal Load on weld	$H_w := V_d$	$H = 2.1 \text{ kip}$	
Weld electrode strength (E70XX)	$F_w := 0.3 \cdot 70 \cdot \text{ksi}$	$F_w = 21.0 \text{ ksi}$	
Weld size	$t_w := \frac{1}{8} \cdot \text{in} \cdot \cos(45 \cdot \text{deg})$		
Length of Weld	$d_w := 3.5 \cdot \text{in}$	$d = 3.5 \text{ in}$	
Weld stress capacity	$W_{\text{cap}} := t_w \cdot F_w \cdot 2 \cdot d$	$W_{\text{cap}} = 13.0 \text{ kip}$	
Vertical Weld Shear stress	$fw_1 := \frac{V}{t_w \cdot 2 \cdot d}$	$fw_1 = 12.3 \text{ ksi}$	
Horizontal Weld Shear stress	$fw_2 := \frac{H}{t_w \cdot 2 \cdot d}$	$fw_2 = 3.5 \text{ ksi}$	
Combine the two stresses	$F_{\text{tot}} := \frac{fw_2}{2} + \sqrt{\left(\frac{fw_1}{2}\right)^2 + \left(\frac{fw_2}{2}\right)^2}$	$F_{\text{tot}} = 14.3 \text{ ksi}$	
	Safety Factor	$\frac{F_w}{F_{\text{tot}}} = 1.471 > 1.0 \text{ is OK}$	

### Concrete Resistance to Pull-out Forces.

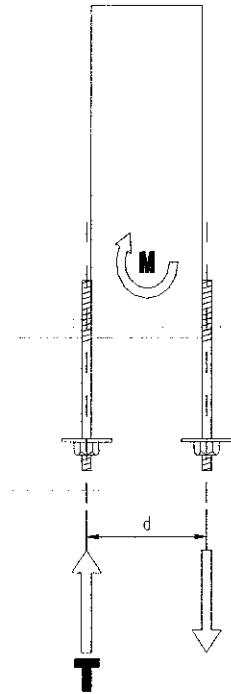
Determine Cone surface area. One Cone pr. Bolt.

Moment in Column Base  $M_d := M_t$   $M_d = 48.5 \text{ in-kip}$

Anchor bolt Spacing  $d := 8 \text{ in}$

Number of Anchor Bolts in Tension  $N_{ob} := 2$

Anchor Bolt Tension  $T := \frac{M_d}{N_{ob} \cdot d}$   $T = 3.0 \text{ kip}$



### Concrete Resistance to Pull-out Forces.

Determine Cone surface area. One Cone pr. Bolt.

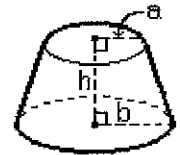
Bolt diameter  $d_b := 0.75 \text{ in}$

Allowable Bolt Tension (A-307)  $T_a := 8.8 \text{ kip}$

Embedment depth:  $h := 4 \text{ in}$

Out-To-Out Bolt spacing:  $sp := 8 \text{ in}$   $c := \frac{sp}{(N_{ob} - 1)} = 8.0 \text{ in}$

Radius of the base of the Fustrum:  $a := \frac{d_b}{2}$   $a = 0.375 \text{ in}$



Force propagation is 1:2, Radius of top of concrete Cone

$b := a + \frac{h}{2}$   $b = 2.4 \text{ in}$   $b < \frac{c}{2}$

Find the lateral surface area pr bolt:

$Ar := \pi \cdot (a + b) \cdot \sqrt{h^2 + (b - a)^2}$   $Ar = 38.6 \text{ in}^2$

**Pull-out force Resistance:**

$f_{v_c} := 4 \cdot \sqrt{\frac{f'_c}{\text{ksi}}} \cdot \text{ksi}$   $P_u := f_{v_c} \cdot Ar$   $P_u = 289.1 \text{ kip}$

Safety Factor against Pull-out Force:

$\frac{P_u}{T} = 95.304 > 1.5 \text{ is OK}$

**Load Adjustment Factors**  
for Anchor Spacing

$s_{min} := 0.5 \cdot h$   $s_{cr} := 2 \cdot h$   $s_{cr} = 8.0 \text{ in}$

for Tension and Shear  $\phi_a := \text{if} \left( s_{min} \leq c \leq s_{cr}, 0.2 \cdot \frac{c}{h} + 0.6, 0 \right)$   $\phi_a = 1.00$

Allowable Bolt Tension  $T' := T_a \cdot \phi_a$   $T' = 8.8 \text{ kip}$

$\frac{T'}{T} = 2.901 > 1.0 \text{ is OK}$