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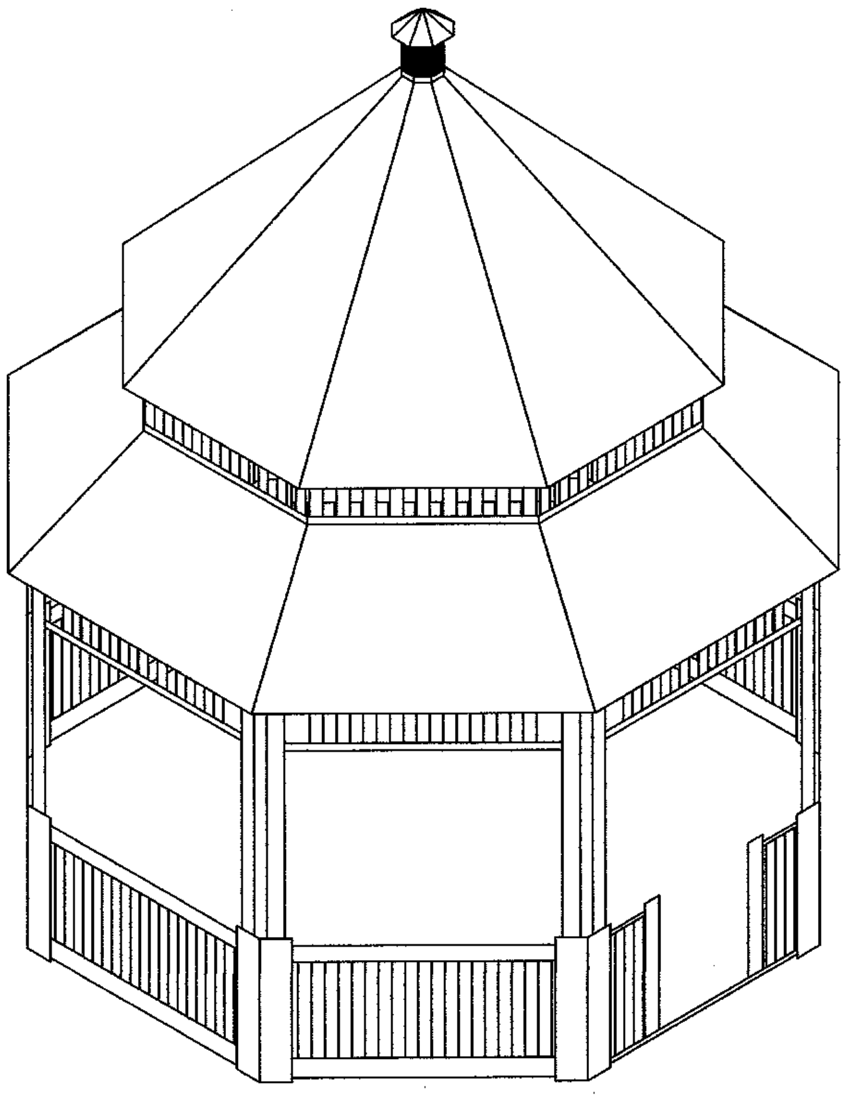
STRUCTURAL ANALYSIS
MODEL 21' PAVILION



LACEY SUBDIVISON
FORKED RIVER, NJ

COMMONWEALTH OF VIRGINIA
GRAHAM HENDERSON
4/4/12
LICENSE No.
4031
PROFESSIONAL ENGINEER

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 - 21' Pavilion**

CALCULATION INDEX

1.1 Useful Units Defined

$plf := \frac{lbf}{ft}$ $psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000 \cdot lbf$ $ksi := \frac{kip}{in^2}$ $ksf := \frac{kip}{ft^2}$

2.1 MATERIALS

Steel - A36	Yield Stress	$Fy_{A36} := 36 \cdot ksi$	Modulus of Elasticity	$E := 29000 \cdot ksi$
	Ultimate stress	$Fu_{A36} := 58 \cdot ksi$		
	Density	$\rho_{A36} := 490 \cdot pcf$		

Wood **Select Structural Western Cedar**

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

Density

Assumed Water Content $mc := 22 \cdot \%$

Specific Gravity $G := 0.36$

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

Concrete

Compressive Strength (AE) $f'_c := 3500 \cdot psi$

Modulus of Elasticity $E_c := 57000 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi$

$E_c = 3.4 \times 10^6\,psi$

Reinforcing Steel Yield stress $Fy_c := 60 \cdot ksi$

Density : $\rho_{con} := 150 \cdot pcf$

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 $B := 21 \cdot ft$

Structure overall height $h := 25 \cdot ft$

Effective Structure Height $z' := 0.6 \cdot h$ $z' = 15.0\,ft$

Terrain Exposure Constant B $z_{min} := 15 \cdot ft$
(ASCE 6.5.6.3 Page 25)

Equivalent structure height $z := if(z' > z_{min}, z', z_{min})$ $z = 15.0\,ft$
(ASCE Page 78)

Horizontal building dimension measured parallel to wind direction $L := 21 \cdot ft$

Surface Roughness C, Building Exposure Category C.

Basic Wind Speed (Forked River, NJ (V mph) $V := 110$
(ASCE page 33)

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

Building Occupancy Category $Cat := 2$
(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 $z := h$ $z = 25.0\,ft$

$$\boxed{0.85 + \frac{0.90 - 0.85}{20 \cdot ft - 15 \cdot ft} \cdot (z - 15 \cdot ft) = 0.950} \qquad K_z := 0.94$$

Interpolation of Table 6-3 for K_z

Topographic Effects K_{zt}
ASCE Figure 6-4, page 45

Height of hill or escarpment relative to the upwind terrain $H := 35 \cdot ft$

Distance Upwind/Downwind to crest to building site $x := 200 \cdot ft$

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

$\lambda_{K1} := \frac{H}{L_h}$ $\lambda_{K1} = 0.23$ $\boxed{0.17 + \frac{0.21 - 0.17}{0.25 - 0.20}\cdot(\lambda_{K1} - 0.20) = 0.197}$ $K_1 := 0.197$

Interpolation of Figure 6-4 for K_1

$\lambda_{K2} := \frac{x}{L_h}$ $\lambda_{K2} = 1.33$ $\boxed{0.75 + \frac{0.63 - 0.75}{1.50 - 1.00}\cdot(\lambda_{K2} - 1.00) = 0.670}$ $K_2 := 0.670$

Interpolation of Figure 6-4 for K_2

$\lambda_{K3} := \frac{z}{L_h}$ $\lambda_{K3} = 0.17$ $\boxed{0.74 + \frac{0.55 - 0.74}{0.20 - 0.10}\cdot(\lambda_{K3} - 0.10) = 0.613}$ $K_3 := 0.708$

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) $q_z := 0.00256\cdot psf\cdot K_z\cdot K_{zt}\cdot K_d\cdot V^2\cdot I$ $q_z = 29.6\,psf$

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 $n_1 := 1000\cdot Hz$

Peak factor for resonant response $g_R := \sqrt{2\cdot\ln(3600\cdot sec\cdot n_1)} + \frac{0.577}{\sqrt{2\cdot\ln(3600\cdot sec\cdot n_1)}}$ $g_R = 5.6$

Mean hourly wind speed factor $b' := 0.65$ (ASCE page 78, table 6-2, Exposure C)

Mean hourly wind speed power law exponent $\alpha' := \frac{1}{6.5}$ (ASCE page 78), Exposure C

Mean hourly wind speed
(eq. 6-14) $V_z := b'\cdot\left(\frac{z'}{33\cdot ft}\right)^{\alpha'}\cdot V\cdot mph\cdot\frac{88}{60}$ $V_z = 136.2\frac{ft}{sec}$

Resonant response factor part $\mathbf{R_h}$ $\eta := 4.6\cdot n_1\cdot\frac{z'}{V_z}$ $\eta = 506.5$ $R_h := \frac{1}{\eta} - \frac{1}{2\cdot\eta^2}\cdot(1 - e^{-2\cdot\eta})$ $R_h = 0.002$

Resonant response factor part $\mathbf{R_B}$ $\eta := 4.6\cdot n_1\cdot\frac{B}{V_z}$ $\eta = 709.1$ $R_B := \frac{1}{\eta} - \frac{1}{2\cdot\eta^2}\cdot(1 - e^{-2\cdot\eta})$ $R_B = 0.001$

Resonant response factor part $\mathbf{R_L}$ $\eta := 15.4\cdot n_1\cdot\frac{L}{V_z}$ $\eta = 2373.8$ $R_L := \frac{1}{\eta} - \frac{1}{2\cdot\eta^2}\cdot(1 - e^{-2\cdot\eta})$ $R_L = 0.000$

Integral length scale factor $L' := 500\cdot ft$ (ASCE page 78, table 6-2, Exposure C)

Integral length scale power law exponent $\varepsilon := \frac{1}{5.0}$ (ASCE page 78, Exposure C)

Turbulence intensity factor $c := 0.2$ (ASCE page 78, Exposure C)

Intensity of turbulence $I_z := c \cdot \left(\frac{33 \cdot ft}{z}\right)^{\frac{1}{6}}$ $I_z = 0.2$

The integral length scale of turbulence at the equivalent height $L_z := L' \cdot \left(\frac{z'}{33 \cdot ft}\right)^{\varepsilon}$ $L_z = 427.1\,ft$

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 $\mathbf{R_n}$ $R_n := \frac{7.47 \cdot N_1}{\left(1 + 10.3 \cdot N_1\right)^{\frac{5}{3}}}$ $R_n = 0.0007$

The Resonant Response Factor $\beta := 0.01$ $R := \sqrt{\frac{1}{\beta} \cdot R_n \cdot R_h \cdot R_B \cdot \left(0.53 + 0.47 \cdot R_L\right)}$ $R = 0.000$

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) $p' := q_z \cdot \left(G_f \cdot C_p - GC_{pi}\right)$ $p' = 21.2\,psf$

External Pressure Coefficients, **Leeward** wall (ASCE, Figure 6.6, page 49) $\frac{L}{B} = 1.0$ $C_p := -0.5$

Lateral Wind Pressure on leeward wall (eq. 6-17, page 28) $p := q_z \cdot \left(G_f \cdot C_p - GC_{pi}\right)$ $p = -13.2\,psf$

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)
$I := 1.0$

Ground Snow Load
"Forked River, NJ " (ASCE page 85)
$p_g := 20 \cdot psf$

Flat Roof Snow load (eq. 7-1 page 81)
$p_f := 0.7 \cdot C_e \cdot C_t \cdot I \cdot p_g$
$p_f = 15.1\,psf$

Roof slope, run "S" for a rise of 12 $S := 6.93$

Slope width $W := 10 \cdot ft$

Sloped Roof Factor
ASCE page 81
Roof Slope $\theta := \frac{360}{2 \cdot \pi} \cdot atan\left(\frac{S}{12}\right)$ $\theta = 30.0$

Warm Roof Slopes (C_t<1.0)
ASCE page 86
$Cs_{warm} := if\left(\theta \le 30, 1, 1 - \frac{\theta - 30}{70 - 30}\right)$ $Cs_{warm} = 1.000$

Cold Roof Slopes (C_t=1.1)
ASCE page 86
$Cs_{cold1} := if\left(\theta \le 37.5, 1, 1 - \frac{\theta - 37.5}{70 - 37.5}\right)$ $Cs_{cold1} = 1.000$

Cold Roof Slopes (C_t=1.2)
ASCE page 86
$Cs_{cold2} := if\left(\theta \le 45, 1, 1 - \frac{\theta - 45}{70 - 45}\right)$ $Cs_{cold2} = 1.000$

Sloped Roof Factor
$C_s := if\left(C_t \le 1, Cs_{warm}, if\left(C_t = 1.1, Cs_{cold1}, Cs_{cold2}\right)\right)$ $C_s = 1.000$

Dry Sloped Roof Snow Load
$p'_s := C_s \cdot p_f$ (eq. 7-2)
$p'_s = 15.1\,psf$

Rain on Snow Surcharge
ASCE page 83
$p_r := if\left(p'_s \ge 20 \cdot psf \wedge \theta \le \frac{360}{2 \cdot \pi} \cdot atan\left(\frac{W}{50 \cdot ft}\right), 5 \cdot psf, 0 \cdot psf\right)$ $p_r = 0.0\,psf$

if p'_s >= 20psf and roofslope<W,50 use 5psf, if not use 0psf

Wet, Sloped Roof, Snow Load
$p_s := p'_s + p_r$
$p_s = 15.1\,psf$

3.3 Dead Load Contribution.

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

Wall Dead Load $W_{DL} := 1.5 \cdot psf$

3.4 Minimum Roof Live Load.

Tributary area $A_t := 59 \cdot ft^2$

Live Load reduction Factor (R_1)
ASCE, page 10

$R_1 := if\left(A_t \le 200 \cdot ft^2, 1, if\left(200 \cdot ft^2 < A_t < 600 \cdot ft^2, 1.2 - 0.001 \cdot A_t, 0.6\right)\right)$ $R_1 = 1.000$

Roof slope, inches of rise per foot $F := 7$

Live Load reduction Factor (R_2)
ASCE, page 11

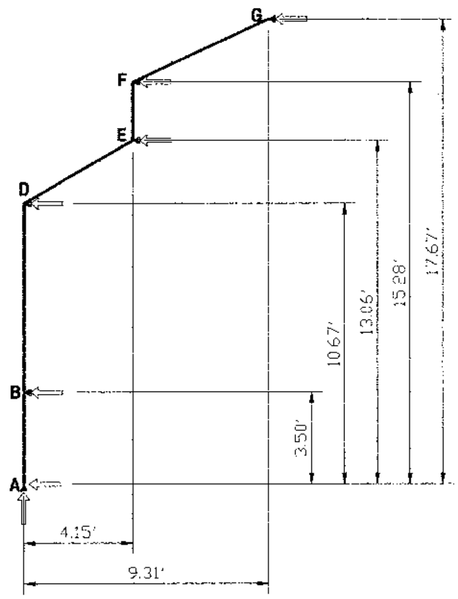
$R_2 := if(F \le 4, 1, if(4 < F < 12, 1.2 - 0.05 \cdot F, 0.6))$ $R_2 = 0.850$

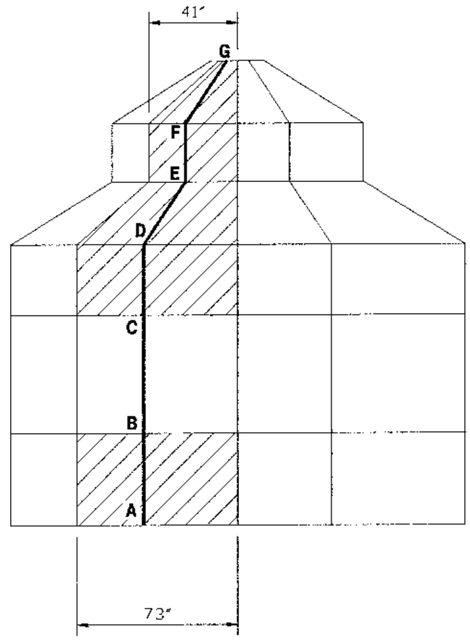
Minimum Roof Live Loads:
ASCE, page 10

$R_{LL} := 20 \cdot psf \cdot R_1 \cdot R_2$ $R_{LL} = 17.0\,psf$

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 — D + 0.75 W + 0.75 S or L
ASCE, page 6

Windward Wind Loads:

Influence width at Base	$B1 := 73\cdot in$	
Influence width at Fret	$B2 := 41\cdot in$	
	$Wind' := 0.75\cdot p'$	$Wind' = 15.9\,psf$
Segment AB	$Wind_{AB} := 0.6\cdot Wind'\cdot B1$	$Wind_{AB} = 58.0\,plf$
Segment BC	$Wind_{BC} := Wind'\cdot 3.5\cdot in$	$Wind_{BC} = 4.6\,plf$
Segment CD	$Wind_{CD} := 0.6 Wind'\cdot B1$	$Wind_{CD} = 58.0\,plf$
Segment DE has a triangular load distribution	$Wind\_A_{DE} := Wind'\cdot B1$	$Wind\_A_{DE} = 96.6\,plf$
	$Wind\_B_{DE} := Wind'\cdot B2$	$Wind\_B_{DE} = 54.3\,plf$
Segment EF	$Wind_{EF} := 0.6\cdot Wind'\cdot B2$	$Wind_{EF} = 32.6\,plf$
Segment FG has a triangular load distribution	$Wind\_A_{FG} := Wind'\cdot B2$	$Wind\_A_{FG} = 54.3\,plf$
	$Wind\_B_{FG} := Wind'\cdot 6\cdot in$	$Wind\_B_{FG} = 7.9\,plf$

Vertical Dead Loads:

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

Vertical Snow or Live:

Segment DE has a triangular load distribution	$Live\_A_{DE} := 0.75 \cdot R_{LL} \cdot B1$	$Live\_A_{DE} = 77.6\,plf$
	$Live\_B_{DE} := 0.75 \cdot R_{LL} \cdot B2$	$Live\_B_{DE} = 43.6\,plf$
Segment FG has a triangular load distribution	$Live\_A_{FG} := 0.75 \cdot R_{LL} \cdot B2$	$Live\_A_{FG} = 43.6\,plf$
	$Live\_B_{FG} := 0.75 \cdot R_{LL} \cdot 6 \cdot in$	$Live\_B_{FG} = 6.4\,plf$

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

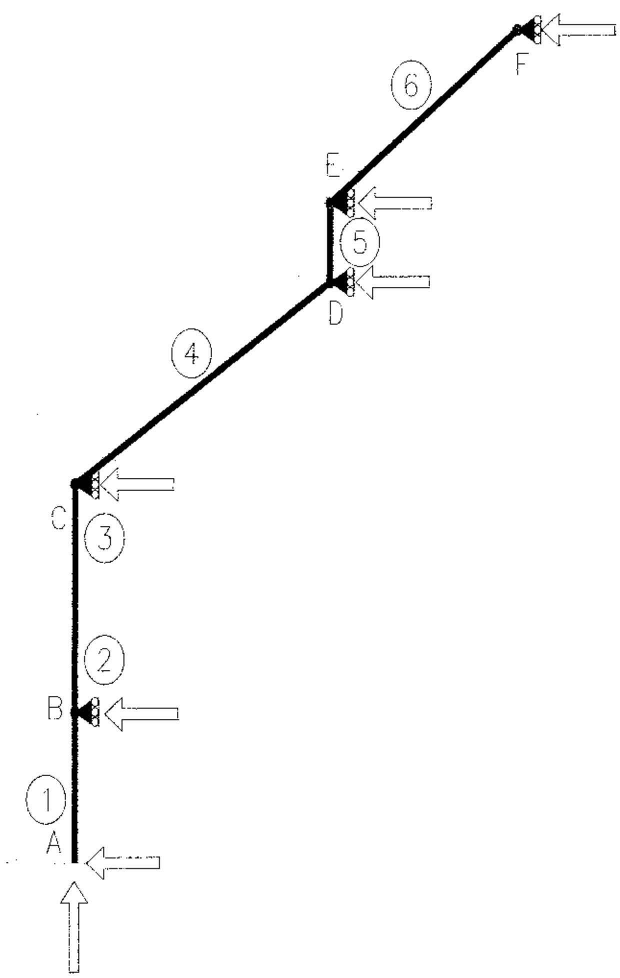
Leeward Direction

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

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 octogonall 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 facia 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 $Rx_B := 42 \cdot lbf$

$Rx_C := 589 \cdot lbf$

$Rx_D := 1061 \cdot lbf$

Vertical Reaction, (N1) RISA 3D $Ry_1 := 2057 \cdot lbf$

Moment in Member 1, (M24) RISA 3D $M_1 := 4045 \cdot ft \cdot lbf$

Moment in Member 2, (M23) RISA 3D $M_2 := 1846 \cdot ft \cdot lbf$

Moment in Member 3, (M22) RISA 3D $M_3 := 2685 \cdot ft \cdot lbf$

Moment in Member 4, (M20) RISA 3D $M_4 := 26 \cdot ft \cdot lbf$

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 $\sigma_1 := 1004 \cdot psi$

Max. Shear Stress $\sigma_v := 55 \cdot psi$

Allowable Shear Stress $F'_v = 75.0\,psi$

Allowable Bending Stress $Fb_w = 1000.0\,psi$

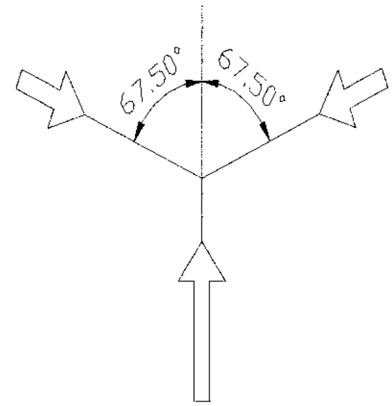
Safety Against Bending Failure $\frac{Fb_w}{\sigma_1} = 1.00$ >=1.0 is **OK**

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

6.0 Component Analysis.

6.1 Analyze Hand Rail

$i := 1..2$



Min. Live Load on Hand rails $\quad W_{hr} := 50 \cdot plf$

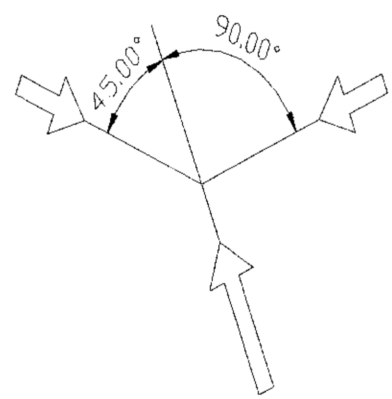
Load decomposition angle with Wind perpendicular to the corner $\quad \alpha_1 := 67.5 \cdot deg$

Load decomposition angle with Wind perpendicular to one surface $\quad \alpha_2 := 45.0 \cdot deg$

Axial force $\quad P2_i := \dfrac{Rx_B}{\dfrac{2}{i} \cdot \cos(\alpha_i)} \qquad P_2 = \begin{pmatrix} 54.9 \\ 59.4 \end{pmatrix} lbf$

X-sect. Area $\quad b := 3.5 \cdot in$

$h := 1.5 \cdot in \quad A := b \cdot h \qquad A = 5.3\,in^2$



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

$L := 48 \cdot in \qquad W := p \cdot 12 \cdot in \cdot 50 \cdot \% \qquad W = -6.6\,plf$

Moment $\quad M2 := \dfrac{(W_{hr} + W) \cdot L^2}{8} \qquad M_2 = 86.8\,ft \cdot lbf$

Section Modulus $\quad S_2 := \dfrac{b \cdot h^2}{6} \qquad S_2 = 1.3\,in^3$

Threaded Rod Diameter $\quad d_r := \dfrac{1}{2} \cdot in$ $\quad$ <u>**Use 1/2" Threaded Rod in all wall Panels**</u>

Rod Area $\quad A_r := \pi \cdot \dfrac{d_r^2}{4} \qquad A_r = 0.2\,in^2$

Rod Section Mod $\quad S_r := \pi \cdot \dfrac{d_r^3}{32} \qquad S_r = 0.012\,in^3$

Steel Modulus of Elasticity $\quad E := 29000 \cdot ksi \quad$ Steel to Wood strength Ratio $\quad n := \dfrac{E}{E'} \qquad n = 29.0$

Check combined Compression and Bending

Max. stress $\quad f_{max_i} := \dfrac{P2_i}{A + n \cdot A_r} + \dfrac{M_2}{S_2 + n \cdot S_r} \qquad f_{max} = \begin{pmatrix} 629.1 \\ 629.5 \end{pmatrix} psi$

Safety Factor $\quad \dfrac{Fb_w}{f_{max}} = \begin{pmatrix} 1.59 \\ 1.589 \end{pmatrix} \quad$ > 1.0 so **OK**

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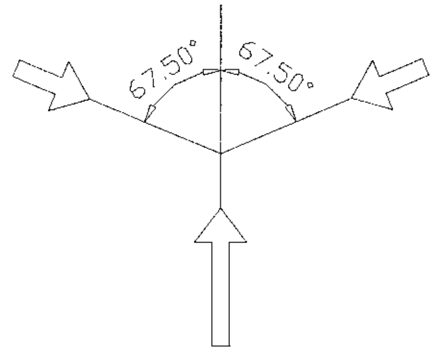
6.2 Analyze Roof Fret



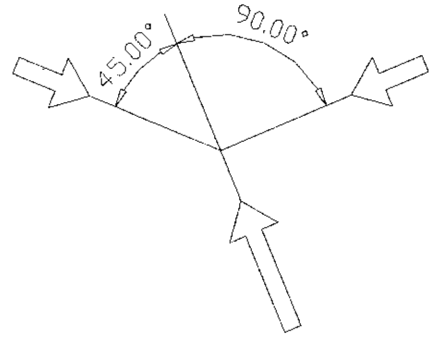
Load decomposition angle with Wind perpendicular to the corner $\alpha_1 := 67.5\cdot deg$

Load decomposition angle with Wind perpendicular to one surface $\alpha_2 := 45.0\cdot deg$

Axial force $i := 1..2$ $P3_i := \dfrac{Rx_C}{\dfrac{2}{i}\cdot\cos(\alpha_i)}$ $P_3 = \begin{pmatrix} 769.6 \\ 833.0 \end{pmatrix} lbf$

Bending Moment due to Vertical loads

$L := 48\cdot in$



Dead Load $W_{dl} := \dfrac{R_{DL}\cdot B\cdot(0.71\cdot 3 + 0.3)}{4}$ $W_{dl} = 159.5\,plf$

Wind Load $W_{wl} := \dfrac{p'\cdot\cos(30\cdot deg)^2\cdot B\cdot(0.71\cdot 3 + 0.3)}{4}$ $W_{wl} = 202.6\,plf$

Combined Vertical Load $W := W_{dl} + W_{wl}$ $W = 362.1\,plf$

Moment $M_3 := \dfrac{W\cdot L^2}{8}$ $M_3 = 724.1\,ft\cdot lbf$

Number of Components $N := 2$ Counter $n := 1..N$

Horizon. dim. of Bot $x_1 := 3.5\cdot in$ Vertical dim of Shape 1 $y_1 := 1.5\cdot in$ Distance to c.g. $d_1 := \dfrac{y_1}{2}$

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

Overall Height of this section $H := 13.75\cdot in$

Determine Location of Neutral Axis of Lower Roof Fret

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

Gross Area $At := \sum_n At'_n$ $At = 10.5\,in^2$

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

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

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

Section Modulus Top $Sf_1 := \dfrac{It}{H - D_{na}}$ Bottom $Sf_2 := \dfrac{It}{D_{na}}$ $Sf = \begin{pmatrix} 57.6 \\ 57.6 \end{pmatrix} in^3$

Check combined Compression and Bending

Max. stress $f_{max_i} := \dfrac{P3_i}{At} + \dfrac{M_3}{Sf_1}$ $f_{max} = \begin{pmatrix} 224.2 \\ 230.2 \end{pmatrix} psi$ Safety Factor $\dfrac{Fb_w}{f_{max}} = \begin{pmatrix} 4.46 \\ 4.343 \end{pmatrix}$ > 1.0 so **OK**

7.1 Safety Against Sliding when built on Concrete Slab.

Effective Projected Area of 21' Pavilion

Area of Hand rails $Ag_1 := 237\cdot in\cdot 42\cdot in\cdot 50\cdot\%$ $Ag_1 = 34.6\,ft^2$ DST to center of g. $d_1 := 28\cdot in$

Area of Low Fret $Ag_2 := 237\cdot in\cdot 14\cdot in\cdot 50\cdot\%$ $Ag_2 = 11.5\,ft^2$ Dist to center of g. $d_2 := 117\cdot in$

Area of Low Roof $Ag_3 := \dfrac{(264\cdot in + 177\cdot in)}{2}\cdot 44\cdot in$ $Ag_3 = 67.4\,ft^2$ Dist to center of g. $d_3 := 140\cdot in$

Area of Riser $Ag_4 := 177\cdot in\cdot 18\cdot in\cdot 50\%$ $Ag_4 = 11.1\,ft^2$ Dist to center of g. $d_4 := 170\cdot in$

Area of High Roof $Ag_5 := \dfrac{(191\cdot in + 13\cdot in)}{2}\cdot 52\cdot in$ $Ag_5 = 36.8\,ft^2$ Dist to center of g. $d_5 := 196\cdot in$

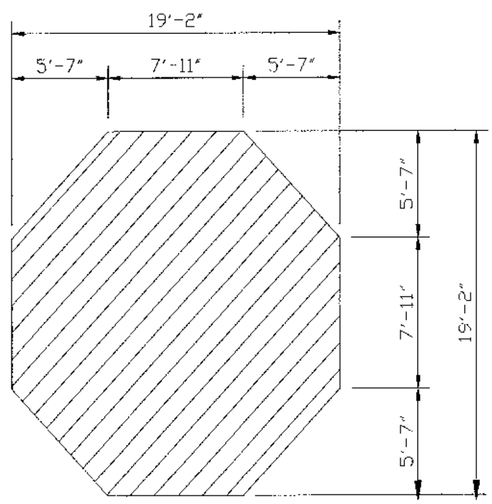
Total projected Area $Ag_{tot} := \sum Ag$ $Ag_{tot} = 161.4\,ft^2$

Lateral Wind Force $V_{tot} := (p' + p)\cdot Ag_{tot}$ $V_{tot} = 1281.2\,lbf$

Assume some common soil parameters

Angle if Internal Friction (estimated) $\theta := 15\cdot deg$

Coefficient of Cohesion (estimated) $c := 0.2\cdot ksf$



Concrete Slab parameters

Slab thickness $t_{slab} := 8\cdot in$

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

$A_{slab} := (230\cdot in)^2 - 2\cdot(67\cdot in)^2$ $A_{slab} = 305.0\,ft^2$

Load combination Wind + 0.6 Dead Load

Weight of Concrete Slab $W_{conc} := 0.6\cdot t_{slab}\cdot A_{slab}\cdot \rho_{con}$ $W_{conc} = 18300.8\,lbf$

Determine the Friction-cohesion resistance $Fr := W_{conc}\cdot\tan(\theta) + \dfrac{2}{3}\cdot c\cdot A_{slab}$ $Fr = 45572.2\,lbf$

Safety factor $\dfrac{Fr}{V_{tot}} = 35.6$ > 1.5 is **OK**

7.2 Safety Against Overturning when built on Concrete Slab

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

Stabilizing Moment due to concrete slab

$B := 230\cdot\text{in}$ $M_s := W_{conc}\cdot\frac{B}{2}$ $M_s = 175383\,\text{ft}\cdot\text{lbf}$

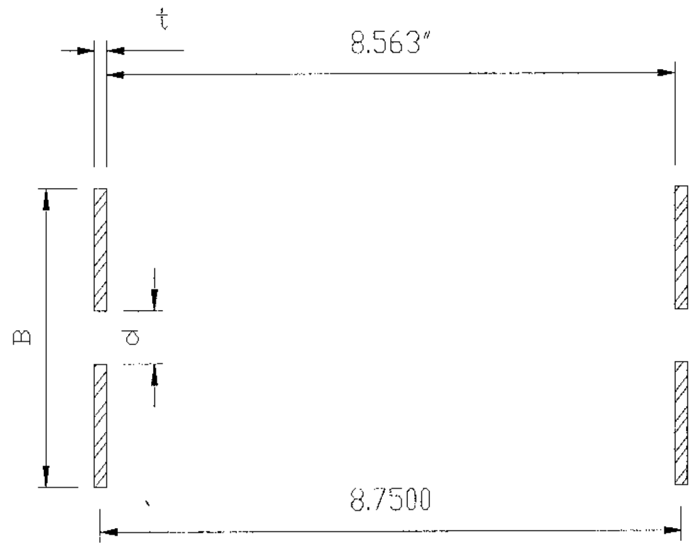
Safety factor $\frac{M_s}{M_o} = 2.9$ > 2.0 is **OK**

8. Column Base Strap Design.

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

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

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



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

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

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

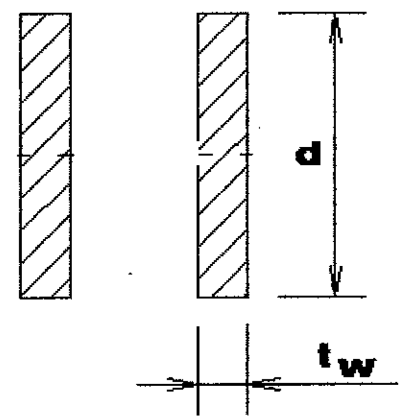
Section Modulus $S_h := \frac{2\cdot I_h}{5.75\cdot\text{in}}$ $S_h = 7.186\,\text{in}^3$

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

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

Allowable Stress in Bracket $F_{all} := 0.6\cdot Fy_{A36}$ $F_{all} = 21.6\,\text{ksi}$

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



Moment imposed on weld	$M := M_d$	$M = 48.5\ \text{in}\cdot\text{kip}$
Vertical Load on weld	$V := 2.07\cdot\text{kip} + \dfrac{M_d}{8.75\cdot\text{in}}$	$V = 7.6\ \text{kip}$
Horizontal Load on weld	$H := V_d$	$H = 2.1\ \text{kip}$
Weld electode strength (E70XX)	$Fw := 0.3\cdot 70\cdot\text{ksi}$	$Fw = 21.0\ \text{ksi}$
Weld size	$t_w := \dfrac{1}{8}\cdot\text{in}\cdot\cos(45\cdot\text{deg})$	
Length of Weld	$d := 3.5\cdot\text{in}$	$d = 3.5\ \text{in}$
Weld stress capacity	$W_{cap} := t_w\cdot Fw\cdot 2\cdot d$	$W_{cap} = 13.0\ \text{kip}$
Vertical Weld Shear stress	$fw_1 := \dfrac{V}{t_w\cdot 2\cdot d}$	$fw_1 = 12.3\ \text{ksi}$
Horizontal Weld Shear stress	$fw_2 := \dfrac{H}{t_w\cdot 2d}$	$fw_2 = 3.5\ \text{ksi}$

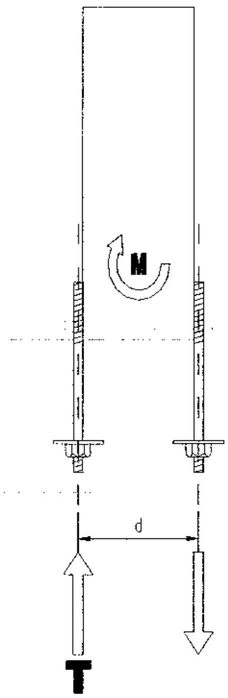
Combine the two stresses

$$F_{tot} := \frac{fw_2}{2} + \sqrt{(fw_1)^2 + \frac{(fw_2)^2}{2}} \qquad F_{tot} = 14.3\ \text{ksi}$$

Safety Factor $\dfrac{Fw}{F_{tot}} = 1.471$ > 1.0 is **OK**

Concrete Resistance to Pull-out Forces.

Determine Cone surface area. One Cone pr. Bolt.



Moment in Column Base $M_d := M_1$ $M_d = 48.5\,\text{in}\cdot\text{kip}$

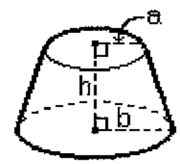
Anchor bolt Spacing $d := 8\text{in}$

Number of Anchor Bolts in Tension $No_b := 2$

Anchor Bolt Tension $T := \dfrac{M_d}{No_b \cdot d}$ $T = 3.0\,\text{kip}$

Concrete Resistance to Pull-out Forces.

Determine Cone surface area. One Cone pr. Bolt.



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

Allowable Bolt Tension (A-307) $T'_a := 8.8\cdot\text{kip}$

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

Out-To-Out Bolt spacing: $sp := 8\cdot\text{in}$ $c := \dfrac{sp}{(No_b - 1)}$ $c = 8.0\,\text{in}$

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

Force propagation is 1:2, Radius of top of concrete Cone $b := a + \dfrac{h}{2}$ $b = 2.4\,\text{in}$ $b < \dfrac{c}{2}$

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: $fv_c := 4\cdot\sqrt{\dfrac{f'_c}{\text{ksi}}}\cdot\text{ksi}$ $Pu := fv_c \cdot Ar$ $Pu = 289.1\,\text{kip}$

Safety Factor against Pull-out Force: $\dfrac{Pu}{T} = 95.304$ > 1.5 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 $\phi_a := \text{if}\left(s_{min} \le c \le s_{cr}, 0.2\cdot\dfrac{c}{h} + 0.6, 0\right)$ $\phi_a = 1.00$

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

$\dfrac{T'}{T} = 2.901$ > 1.0 is **OK**