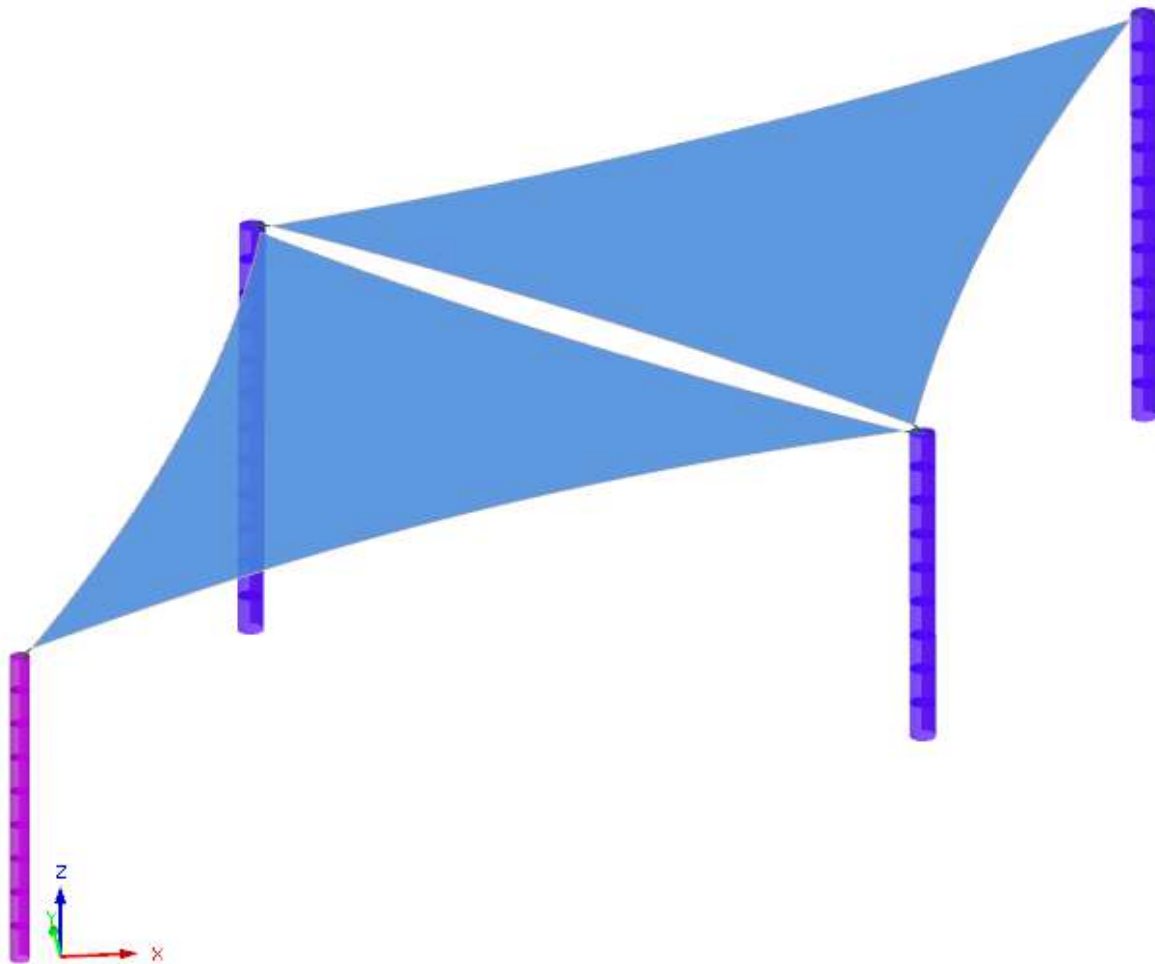
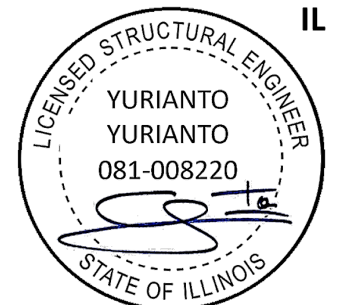


STRUCTURAL CALCULATIONS PACKAGE

Client : Shade Systems
Project No : 10305-1
Project Name : Ty Warner Park
Location : 700 Blackhawk Drive Westmont IL 60559
Subject : Custom Sail SAIL



Prepared By:
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MSC Project No : SS26005
Date: 1/29/2026
Rev: Final



By Yuri at 3:40:28 AM, 2/4/2026
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Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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		Date:	1/29/2026	Final

TABLE OF CONTENTS

	Page
COVER PAGE	1
TABLE OF CONTENTS	2
ASCE 7 LOADS	3
DESIGN CRITERIA	4
LOAD CALCULATIONS	5 to 19
STRUCTURAL ANALYSIS & DESIGN CHECKS	20 to End



IL

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Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Design Codes

Address: 700 Blackhawk Drive Westmont IL 60559

IL

Permanent Structure

Building Code: IBC 2018
ASCE 7-16

IBC 2018
ASCE 7-16

<https://codes.iccsafe.org/codes/i-codes>
<https://asce7hazardtool.online/>

Design load code

Wind Speed, V_{ULT} **105** mph

Ground Elevation

730 ft from sea level

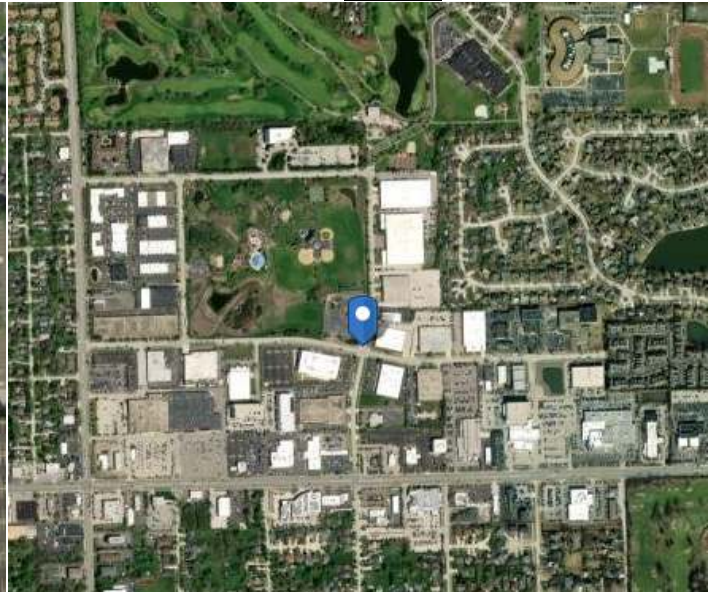
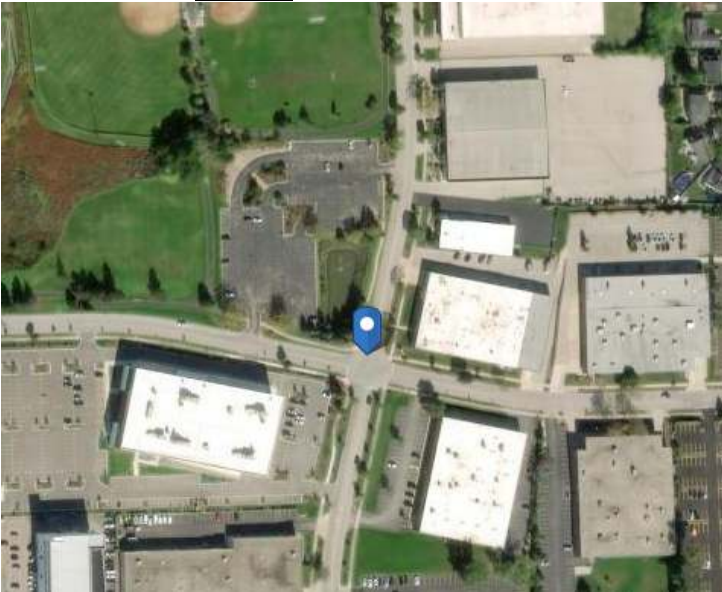
Risk Category

I

V_{ASD} 81 mph

Wind Exposure Category

C



Seismic

Site Class	D
S_s	0.129 g
S_1	0.065 g
0.206 S_{MS}	0.206 g
0.156 S_{M1}	0.156 g

SDC = **B**
(Seismic Design Category)

Ground Snow p_g = **25.00** psf
Non Reducible Roof Snow **n/a**

Ground Snow Load, p_g 25 lb/ft²

S_s	0.129	S_1	0.065
S_{MS}	0.206	S_{M1}	0.156
T_L	12	PGA	0.066
I_e	1	C_v	0.7
F_a	1.6	F_v	2.4
S_{DS}	0.137	S_{D1}	0.104
PGA_M	0.105	F_{PGA}	1.6

Seismic Design Category **B**

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

DESIGN CRITERIA

Address 700 Blackhawk Drive Westmont IL 60559

Permanent Structure

Design Codes IBC 2018 IBC 2018
ASCE 7-16 ASCE 7-16

Roof Live Load

Roof construction supported by a skeleton structure.

$L_{o\text{ roof}} = 5 \text{ psf}$ $L_{\text{conc}} = 300 \text{ lb}$

Snow Load

Roof Type Monoslope
Ground snow loads $p_g = 25.00 \text{ psf}$
Terrain category C
Exposure of roof Fully Exposed
Risk category I
Thermal factor $C_t = 1.10$
Exposure factor $C_e = 0.9$
Importance factor for snow loads $I_s = 0.80$
Roof surface type All Other Surfaces
Roof slope factor $C_s = 1.000$
Flat roof snow $p_f = 16.00 \text{ psf}$
Sloped roof snow Load $p_s = 16.00 \text{ psf}$
Non reducible roof snow n/a n/a

Fabric should be removed when a snow event is expected

Wind Load

Risk category I
Wind exposure category C
Basic wind speed, 3-sec gust speed over land $V_{ULT} = 105 \text{ mph}$
Table C26.5-2 Saffir-Simpson Hurricane Category 1
Ground elevation above sea level $z_g = 730 \text{ ft}$
Wind importance factor $I_w = 1.00$
Gust-effect factor $G = 0.85$
Velocity Pressure Exposure Coefficient $K_h = 0.85$
Ground Elevation Factor $K_e = 0.97$
Wind Directional Factor $K_d = 0.85$
Internal Pressure Coefficient $(G C_{pi}) = +/- 0.00$
Topographic factor $K_{zt} = 1.0$
Mean roof velocity pressure $q_h = 14.57 \text{ psf}$

The fabric is readily removable and needs to be removed if wind speeds exceeding 90 mph are expected.

$V_{ASD} = 81 \text{ mph}$

Wind Speed = 95 mph

Seismic Load

Soil site classification D
Soil type Stiff soil
Risk Category I
Importance factor $I_e = 1.00$
Short period ground acceleration $S_s = 0.129 \text{ g}$
1-second ground acceleration $S_1 = 0.065 \text{ g}$
Short period site coefficient $F_a = 1.600$
Long period site coefficient $F_v = 2.400$
For short periods, $F_a S_s$ $S_{MS} = 0.206 \text{ g}$
For 1-second period, $F_v S_1$ $S_{M1} = 0.156 \text{ g}$
For short periods, $2/3 S_{MS}$ $S_{DS} = 0.138 \text{ g}$
For 1-second period, $2/3 S_{M1}$ $S_{D1} = 0.104 \text{ g}$
Seismic Design Category B

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

LOAD COMBINATIONS

Design Codes IBC 2018
ASCE 7-16

IBC 2018
ASCE 7-16

<u>Load Types</u>	<u>ASD Load Combinations</u>	<u>LRFD Load Combinations</u>
D Dead	D	1.4D
L Live	D + L	1.2D + 1.6L
Lr Roof Live	D + Lr	1.2D + 1.6L + 0.5Lr
S Snow	D + S	1.2D + 1.6L + 0.5S
E Earthquake	D + 0.75L + 0.75Lr	1.2D + L + 1.6Lr
W Wind	D + 0.75L + 0.75S	1.2D + L + 1.6S
	D + 0.7E	1.2D + 1.6Lr
	D + 0.6W	1.2D + 1.6Lr + 0.5W
	D + 0.75L + 0.75Lr + 0.45W	1.2D + 1.6S
	D + 0.75L + 0.75S + 0.45W	1.2D + 1.6S + 0.5W
	0.6D + 0.6W	1.2D + L + 0.5Lr + W
	0.6D + 0.7E	1.2D + L + 0.5S + W
		1.2D + L + 0.2S + E
		0.9D + W
		0.9D + E

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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		Date:	1/29/2026	Final

LIVE LOAD

Roof Live Load

[Section 4.8, pg. 17]

Roof ID	Main Roof			
Design load code	ASCE 7-16	ASCE 7-16		
Roof type	Canopies - Fabric construction supported by a skeleton structure.			
Roof shape	Flat or Pitched			
Pitched roof angle	8.89	$\theta =$	8.89	degrees .
Min. Uniform load	5 No	$L_{o\ roof} =$	5	psf Reduction permitted ? No [Table 4.3-1, pg. 15]
Min. Concentated load		$L_{conc} =$	300	lb 300 lb

Note: Uniform load is to be applied on horizontal projection supported by the member.
 Concentrated load is to be applied to skeleton structure
 Concentrated load shall be located so as to produce the max. load effects in the members

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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		Date:	1/29/2026	Final

SNOW LOAD

Note : Fabric should be removed when a snow event is expected

Design load code ASCE 7-16 **ASCE 7-16** [ASCE 7-16, Chap. 7. pg. 51]

Roof Type Monoslope **Monoslope**

Ground snow loads 25.00 $P_g = 25.00$ psf +/- 17.4 inches [Fig. 7.2-1. pg. 52]

Horizontal distance from eave to ridge 19.17 $W = 19.17$ ft (Typically 1/2 L)

Roof angle 8.89 $\theta = 8.89$ degrees

Terrain category C **C** [Table 7.3-1., pg. 58]

Exposure of roof **Fully Exposed** [Table 7.3-1., pg. 58]

Risk category I **I** [Table 1.5-1., pg. 4]

Thermal factor $C_t = 1.10$ Warm roofs: $C_t = 0.85$ or $C_t = 1.0$
Cold roofs: $C_t = 1.1$ or $C_t = 1.2$
Below freezing: $C_t = 1.3$ [Table 7.3-2. pg. 58]

Exposure factor $C_e = 0.9$ [Table 7.3-1., pg. 58]

Importance factor for snow loads 0.80 $I_s = 0.80$ [Table 1.5-2. pg. 5]

Roof surface type **All Other Surfaces** [Section 7.4. pg. 54]

Roof slope factor $C_s = 1.000$ [Fig. 7.4-1. pg. 59]

Snow density $\gamma = 0.13 p_g + 14 \leq 30$ pcf
 $= 17.3$ pcf

Permanent Structure 1.00 1.00 Reduction factor
Not Applicable

Roof Slope, θ	C_s
37.5	1
8.89	1.88
70	0

$C_{s \text{ approx.}} = 1.000$

Non Reducible Roof Snow $p_s = \text{n/a}$ psf

Flat Roof Snow Loads $p_f = 0.7 C_e C_t I_s p_g$ $p_f = 13.86$ psf [Eq. 7.3-1, pg. 51]

p_f min for roof slope < 15 degrees 16.00 $p_m = 16.00$ psf [7.3.4., pg. 53]

25.00
Flat roof snow 16.00 $p_f = 16.00$ psf +/- 11.1 inches

Sloped Roof Snow Loads [7.4., pg. 54]

Balanced Snow Load $p_s = C_s p_f$ 16.00 $p_s = 16.00$ psf +/- 11.1 inches [Eq. 7.4-1, pg. 54]

Unbalanced Snow Load $I_s p_g = 20.00$ psf
 $W \leq 20\text{ft}$, with roof rafter system

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Unbalanced Snow Load , Other [7.6, pg. 57]

Roof slope run for a rise of one, $S = 1 / \tan(\theta)$ $S = 6.4$

Snow density $\gamma = 0.13 p_g + 14 \leq 30$ pcf [Eq. 7.7-1, pg. 59]

$0.13 p_g + 14 = 17.25$ pcf < 30 pcf

Use $\gamma = 17.25$ pcf

Eave to ridge distance, $l_u = W$ $l_u = 19.17$ ft $l_{u \min} = 20$ ft [7.6.1, pg. 57]

For windward portion of the roof

Use $l_u = 20.00$ ft For Fig. 7-9 at pg. 41

Height of snow drift $h_d = 0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10} - 1.5$ [Eq. for Fig. 7.6-1, pg. 61]

$h_d = 1.34$ ft

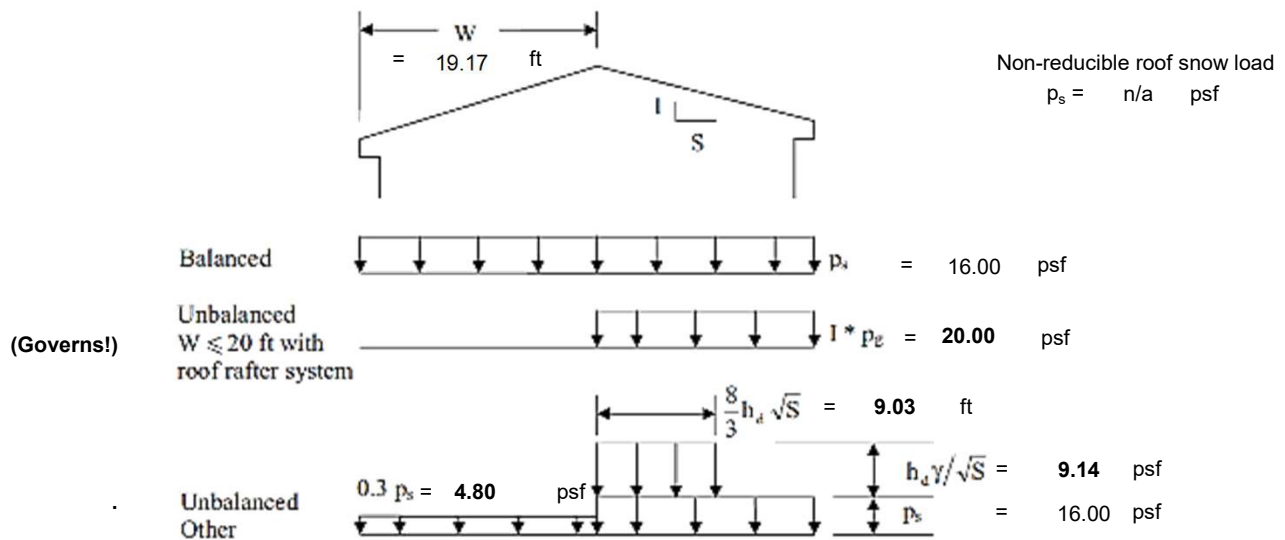
Leeward surcharge snow load length from ridge $\left(\frac{8}{3}\right) h_d \sqrt{S} = 9.03$ ft

Leeward surcharge snow load magnitude $h_d \gamma / \sqrt{S} = 9.14$ psf

Windward unbalanced snow load magnitude $0.3 p_s = 4.80$ psf

Summary of snow load

[Fig. 7.6-2, pg. 61]



Note: Unbalanced snow loads need not be considered for $\theta > 30.2^\circ$ (7 on 12) or for $\theta < 2.38^\circ$ (1/2 on 12)

Roof angle $\theta = 8.89$ degrees
Unbalanced snow loads need to be considered

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WIND LOAD	ID	Frame with Fabric						
Design load code	ASCE 7-16	ASCE 7-16			[Table 26.6-1, pg. 266]			
Structural type		Buildings - Main Wind Force Resisting System						
Roof type		Monoslope						
Risk category	I	I			[Table 1.5-1, pg. 4]			
Enclosure classification		Open buildings			[Table 26.13-1, pg. 271]			
Wind exposure category	C	C			[Section 26.7.3, pg. 266]			
Basic wind speed	105	$V_{ULT} =$	90	mph	$V_{ASD} =$	70	mph	[Figure 26.5-1, pg. 247-249]
Temporary Structure Max.	115							
Building span		$L =$	19.2	ft				
Building length		$B =$	49.4	ft				
Roof peak height		$h_r =$	12.0	ft				
Roof eave height		$z =$	9.0	ft				
Mean roof height	10.5	$h =$	10.5	ft				
Roof slope	8.9	$\theta =$	8.9	degrees	y (ft)	3.00	$\tan^{-1} (y/x)$	
					x (ft)	19.17	8.89	
Ground elevation above sea level	730	$z_g =$	730	ft				
Wind importance factor		$I_w =$	1.00		[Table 1.5-2, pg. 5]			
Gust-effect factor		$G =$	0.85		[Section 26.11.1, pg. 269]			
Wind exposure category		$K_h =$	2.01	$(h / z_g)^{2/\alpha}$	$K_z =$	2.01	$(z / z_g)^{2/\alpha}$	[Table 26.10-1, pg. 268]
		$\alpha =$	9.5		$\alpha =$	9.5		
		$z_g =$	900		$z_g =$	900		
		$K_h =$	0.85		$K_z =$	0.85		
Ground Elevation Factor		$K_e = e^{-0.000362 z_g}$			[Table 26.9-1, pg. 268]			
		$K_e =$	0.97					
Wind Directional Factor	1.00	$K_d =$	0.85		[Table 26.6-1, pg. 266]			
Internal Pressure Coefficient		$(G C_{pi}) =$	+/- 0.00		[Table 26.13-1, pg. 271]			
Topographic factor		$K_{zt} =$	1.0		[Section 26.8.2, pg. 268]			
Permanent Structure	1.00		1.00	Reduction factor	Not Applicable			
Velocity pressure		$q_h =$	14.57	psf	$q_z =$	14.57	psf	[Eq. 26.10-1, pg. 268]
ASCE 7-16		$q_{h (or z)} = 0.00256 K_h (or z) K_{zt} K_d K_e V^2$						

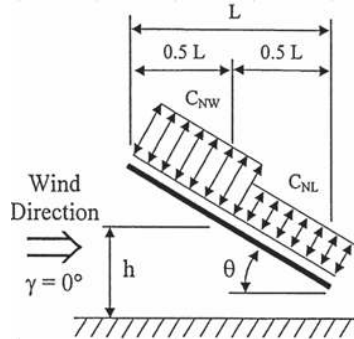
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Net Design Pressures for MWFRS - Pitched Free Roofs

I. Wind direction normal to ridge ($\theta \leq 45^\circ$, $\gamma = 0^\circ$)

$p = q_h G C_N$

Eq. (27.4-3) page 262]



a. Clear Wind Flow

b. Obstructed Wind Flow

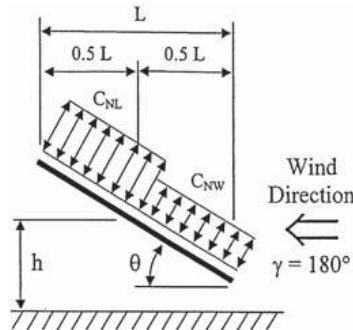
Case A	$p_{NW} = -8.12$ psf	$C_N = -0.66$	Case A	$p_{NW} = -12.62$ psf	$C_N = -1.02$
	$p_{NL} = -13.08$ psf	$C_N = -1.06$		$p_{NL} = -18.58$ psf	$C_N = -1.50$
Case B	$p_{NW} = -18.49$ psf	$C_N = -1.49$	Case B	$p_{NW} = -21.98$ psf	$C_N = -1.77$
	$p_{NL} = 0.00$ psf	$C_N = 0.00$		$p_{NL} = -9.45$ psf	$C_N = -0.76$

Plus and minus signs signify pressures acting toward and away from surfaces, respectively

II. Wind direction normal to ridge ($\theta \leq 45^\circ$, $\gamma = 180^\circ$)

$p = q_h G C_N$

Eq. (27.4-3) page 262]



a. Clear Wind Flow

b. Obstructed Wind Flow

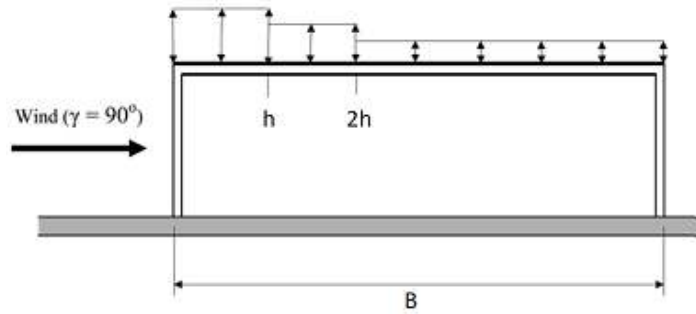
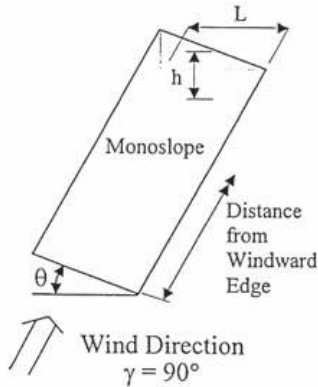
Case A	$p_{NW} = 12.07$ psf	$C_N = 0.97$	Case A	$p_{NW} = -1.10$ psf	$C_N = -0.09$
	$p_{NL} = 18.81$ psf	$C_N = 1.52$		$p_{NL} = -14.63$ psf	$C_N = -1.18$
Case B	$p_{NW} = 20.28$ psf	$C_N = 1.64$	Case B	$p_{NW} = 10.83$ psf	$C_N = 0.87$
	$p_{NL} = 4.41$ psf	$C_N = 0.36$		$p_{NL} = -3.72$ psf	$C_N = -0.30$

Plus and minus signs signify pressures acting toward and away from surfaces, respectively

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
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III. Wind direction parallel to ridge ($\theta \leq 45^\circ$, $\gamma = 90^\circ$)

[Fig. 27.4-7, page 270]



$p = q_h G C_N$ (psf)

Eq. (27.4-3) page 262]

Horizontal Distance from Windward Edge

	$\leq h$ Less or equal to h h = 10.50 ft	$>h, \leq 2h$ More than h, less or equal to 2h	$>2h$ More than 2h 2h = 21.00 ft
a. Clear Wind Flow			
Case A	$p_N = -9.91$ psf $C_N = -0.80$	$p_N = -7.43$ psf $C_N = -0.60$	$p_N = -3.72$ psf $C_N = -0.30$
Case B	$p_N = 9.91$ psf $C_N = 0.80$	$p_N = 6.19$ psf $C_N = 0.50$	$p_N = 3.72$ psf $C_N = 0.30$
b. Obstructed Wind Flow			
Case A	$p_N = -14.86$ psf $C_N = -1.20$	$p_N = -11.15$ psf $C_N = -0.90$	$p_N = -7.43$ psf $C_N = -0.60$
Case B	$p_N = 6.19$ psf $C_N = 0.50$	$p_N = 6.19$ psf $C_N = 0.50$	$p_N = 3.72$ psf $C_N = 0.30$

Plus and minus signs signify pressures acting toward and away from surfaces, respectively

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Design wind loads for Open Frames

[Figure 29.4-2, pg. 325]

Ratio of solid area to gross area	$\epsilon < 0.1$	$\epsilon < 0.1$	$\epsilon < 0.1$
Frame member shape	Flat-Sided Members	Rounded Members	Rounded Members
Diameter of round member		$D \leq 0.655$ ft [7.86"]	$D > 0.656$ ft [7.87"]
Velocity pressure	$q_z = 14.57$ psf	$q_z = 14.57$ psf	$q_z = 14.57$ psf
		$D\sqrt{q_z} = 2.50 < 2.5$	$D\sqrt{q_z} = 2.51 > 2.5$
Gust-effect factor	$G = 0.85$	$G = 0.85$	$G = 0.85$
Force coefficient	$C_f = 2.0$	$C_f = 1.2$	$C_f = 0.8$
Design wind pressure	$p = q_z G C_f$	$p = q_z G C_f$	$p = q_z G C_f$
	$p = 24.77$ psf	$p = 14.86$ psf	$p = 9.91$ psf
	Member ID Member Width (inches) Linear Load (lb / ft)	Member ID Member Width (inches) Linear Load (lb / ft)	Member ID Member Width (inches) Linear Load (lb / ft)
		Column 6.63 8.21	Column 8.63 7.12

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Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Wind Tunnel Procedure

31.1 SCOPE

The Wind Tunnel Procedure shall be used where required by Sections 27.1.3, 28.1.3, and 29.1.3. The Wind Tunnel Procedure shall be permitted for any building or other structure in lieu of the design procedures specified in Chapter 27 (main wind force resisting system [MWFRS] for buildings of all heights and simple diaphragm buildings with $h \leq 160$ ft (48.8 m)), Chapter 28 (MWFRS of low-rise buildings and simple diaphragm, low-rise buildings), Chapter 29 (MWFRS for all other structures), and Chapter 30 (components and cladding [C&C] for all building types and other structures).

User Note: Chapter 31 may always be used for determining wind pressures for the MWFRS and/or for C&C of any building or other structure. This method is considered to produce the most accurate wind pressures of any method specified in this standard.

31.4 LOAD EFFECTS

31.4.1 Mean Recurrence Intervals of Load Effects.

The load effect required for strength design shall be determined for the same mean recurrence interval as for the analytical method by using a rational analysis method, defined in the recognized literature, for combining the directional wind tunnel data with the directional meteorological data or probabilistic models based thereon. The load effect required for allowable stress design shall be equal to the load effect required for strength design divided by 1.6. For buildings or other structures that are sensitive to possible variations in the values of the dynamic parameters, sensitivity studies shall be required to provide a rational basis for design recommendations.

31.4.2 Limitations on Wind Speeds. The wind speeds and probabilistic estimates based thereon shall be subject to the limitations described in Section 26.5.3.

31.4.3 Wind Directionality. The directional wind climate based on recorded or simulated directional wind speed data shall be considered in determining wind loads, and the data shall be presented as part of the wind tunnel report submitted to the Authority Having Jurisdiction. The method for combining wind tunnel model data with information on wind speed and direction at the project site shall also be clearly stated in the Wind Tunnel Report. Variation in the wind direction, based upon uncertainty in the wind climate data, shall be considered when determining the wind loading, and the design wind loads shall be based on the largest values that result from this uncertainty. Consideration of uncertainty in the wind direction is not required in the determination of serviceability related wind effects.

31.4.4 Limitations on Loads. Loads for the MWFRS determined by wind tunnel testing shall be limited such that the overall principal loads in the x and y directions are not less than 80% of those that would be obtained from Part 1 of Chapter 27 or Part 1 of Chapter 28 for buildings or Chapter 29 for other structures. The overall principal load for buildings shall be based on the overturning moment for flexible buildings and the base shear for other buildings. The overall principal load for other structures shall be based on the overturning moment for flexible structures and the base shear for other structures. Pressures for C&C determined by wind tunnel testing shall be limited to not less than 80% of those calculated for Zone 4 for walls and Zone 1 for roofs using the procedure of Chapter 30. These zones refer to those shown in Figs. 30.3-1, 30.3-2A-C, 30.3-3, 30.3-4, 30.3-5A-B, 30.3-6, 30.3-7, and 30.5-1.

The limiting values of 80% may be reduced to 50% for the MWFRS and 65% for C&C if either of the following conditions applies:

1. There were no specific influential buildings or objects within the detailed proximity model.
2. Loads and pressures from supplemental tests for all significant wind directions in which specific influential buildings or objects are replaced by the roughness representative of the adjacent roughness condition, but not rougher than Exposure B, are included in the test results.

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		Date:	1/29/2026	Final

WIND LOAD

ID *Frame without fabric*

Design load code	ASCE 7-16	ASCE 7-16	[Table 26.6-1, pg. 266]
Structural type		Single-Plane Open Frames	
Roof type		Monoslope	
Risk category	I	I	[Table 1.5-1, pg. 4]
Enclosure classification		Open buildings	[Table 26.13-1, pg. 271]
Wind exposure category	C	C	[Section 26.7.3, pg. 266]
Basic wind speed	105	$V_{ULT} = 105$ mph	$V_{ASD} = 81$ mph
Temporary Structure Max.	115		[Figure 26.5-1, pg. 247-249]
Building span		$L = 19.2$ ft	
Building length		$B = 49.4$ ft	
Roof peak height		$h_r = 12.0$ ft	
Roof eave height		$z = 9.0$ ft	
Mean roof height	10.5	$h = 10.5$ ft	
Roof slope	8.9	$\theta = 8.9$ degrees	y (ft) 3.00 $\tan^{-1}(y/x)$ x (ft) 19.17 8.89
Ground elevation above sea level	730	$z_g = 730$ ft	
Wind importance factor		$I_w = 1.00$	[Table 1.5-2, pg. 5]
Gust-effect factor		$G = 0.85$	[Section 26.11.1, pg. 269]
Wind exposure category		$K_h = 2.01 (h / z_g)^{2/\alpha}$ $\alpha = 9.5$ $z_g = 900$ $K_h = 0.85$	$K_z = 2.01 (z / z_g)^{2/\alpha}$ $\alpha = 9.5$ $z_g = 900$ $K_z = 0.85$ [Table 26.10-1, pg. 268]
Ground Elevation Factor		$K_e = e^{-0.000362 z_g}$ $K_e = 0.97$	[Table 26.9-1, pg. 268]
Wind Directional Factor	1.00	$K_d = 0.85$	[Table 26.6-1, pg. 266]
Internal Pressure Coefficient		$(G C_{pi}) = +/- 0.00$	[Table 26.13-1, pg. 271]
Topographic factor		$K_{zt} = 1.0$	[Section 26.8.2, pg. 268]
Permanent Structure	1.00	1.00	Reduction factor Not Applicable
Velocity pressure		$q_h = 19.83$ psf	$q_z = 19.83$ psf [Eq. 26.10-1, pg. 268]
ASCE 7-16		$q_{h (or z)} = 0.00256 K_h (or z) K_{zt} K_d K_e V^2$	

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Design wind loads for Open Frames

[Figure 29.4-2, pg. 325]

Ratio of solid area to gross area	$\epsilon < 0.1$	$\epsilon < 0.1$	$\epsilon < 0.1$
Frame member shape	Flat-Sided Members	Rounded Members	Rounded Members
Diameter of round member		$D \leq 0.561$ ft [6.74"]	$D > 0.562$ ft [6.75"]
Velocity pressure	$q_z = 19.83$ psf	$q_z = 19.83$ psf	$q_z = 19.83$ psf
		$D\sqrt{q_z} = 2.50 < 2.5$	$D\sqrt{q_z} = 2.51 > 2.5$
Gust-effect factor	$G = 0.85$	$G = 0.85$	$G = 0.85$
Force coefficient	$C_f = 2.0$	$C_f = 1.2$	$C_f = 0.8$
Design wind pressure	$p = q_z G C_f$	$p = q_z G C_f$	$p = q_z G C_f$
	$p = 33.72$ psf	$p = 20.23$ psf	$p = 13.49$ psf
	Member ID Member Width (inches) Linear Load (lb / ft)	Member ID Member Width (inches) Linear Load (lb / ft)	Member ID Member Width (inches) Linear Load (lb / ft)
		Column 6.63 11.17	Column 8.63 9.69

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SEISMIC LOAD

ID *Main Building*

Design load code ASCE 7-16 ASCE 7-16 [ASCE 7-16. Chap 11, 12. pg. 77, 89]

Structure height from base to highest level 12.00 $h_n = 12.00$ ft

Soil site classification D D [Table 20.3-1, page 204]

Soil type Stiff soil

Risk Category I I [Table 1.5-1., pg. 4]

Occupancy type Low hazard structures

Importance factor $I_e = 1.00$ [Table 1.5-2, page 5]

Short period ground acceleration 0.129 $S_s = 0.129$ g [Fig. 22-1, page 210-211]

1-second ground acceleration 0.065 $S_1 = 0.065$ g [Fig. 22-2, page 212-213]

Short period site coefficient $F_a = 1.600$ [Table 11.4-1, page 84]

Long period site coefficient $F_v = 2.400$

Permanent Structure 1.00 1.00 Reduction factor Not Applicable

S_s	F_a
< 0.250	1.600
0.129	1.697
0.500	1.400

$F_{a \text{ Interpolation}} = 1.600$

Section 11.4.8. Exception Check
2021 Supplement 3 for ASCE 7-16

1. Site Class D with $S_1 \geq 0.2$ 1.00
2. Site Class E with $S_s \geq 1.0$ or $S_1 \geq 0.2$

[Table 11.4-2, page 84]

S_1	F_v
< 0.100	2.400
0.065	2.470
0.200	2.200

$F_{v \text{ Interpolation}} = 2.400$

Adjusted Maximum Considered Earthquake (MCE) response accelerations [11.4.4, page 84]

For short periods, $F_a S_s$ 0.206 $S_{MS} = 0.206$ g [Eq. (11.4-1)]
For 1-second period, $F_v S_1$ 0.156 $S_{M1} = 0.156$ g [Eq. (11.4-2)]

Design spectral accelerations [11.4.4]

For short periods, $2/3 S_{MS}$ 0.138 **$S_{DS} = 0.138$ g** Section 12.8.1.3 Criteria **not applicable** [Eq. (11.4-3)]
For 1-second period, $2/3 S_{M1}$ $S_{D1} = 0.104$ g [Eq. (11.4-4)]

Seismic Design Category (SDC) [11.6, page 67]

SDC based on S_{DS} 1 A [Table 11.6-1]
SDC based on S_{D1} 2 B [Table 11.6-2]
Check $S_1 \geq 0.75$, Risk Cat. I, II, III 0 (n.a.)
Check $S_1 \geq 0.75$, Risk Cat. IV 0 (n.a.)
Seismic Design Category 2 **B**

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		Date:	1/29/2026	Final

Period determination

[12.8.2, page 102]

Long-period transition period	$T_L = 16.00$ sec		
$T_s = S_{D1} / S_{DS}$	$T_s = 0.76$ sec		[Fig. 22-14, page 225]
Approximate fundamental period	$T_a = C_t h_n^x$		
Maximum value of fundamental period	$T_{max} = T_a C_u$		
Coefficient for upper limit on calculated period	$C_u = 1.7$ (For S_{D1} value see next page)		
Structure height from base to highest level	$h_n = 12.00$ ft		
Steel moment-resisting frames $T_a = 0.028 h_n^{0.8}$	$T_a = 0.204$ sec	$T_{max} = 0.347$ sec	
Eccentrically braced steel frames $T_a = 0.03 h_n^{0.75}$	$T_a = 0.193$ sec	$T_{max} = 0.329$ sec	
All other structural systems $T_a = 0.02 h_n^{0.75}$	$T_a = 0.129$ sec	$T_{max} = 0.219$ sec	

I. Transverse

II. Longitudinal

All other structural systems

All other structural systems

Structural type
Determine structure fundamental periods

T = 0.129 sec

T = 0.129 sec

Allowable Story Drift, Δ_a

[Table 12.12-1]

Risk Cat. All other structures

I or II	0.020	h_{sx}
III	0.015	h_{sx}
IV	0.100	h_{sx}

Risk Cat. I 0.020 $\Delta_a = 0.020 h_{sx}$

Story Height $h_{sx} = 12.00$ ft $\Delta_a = 2.88$ inches

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CHAPTER 12

Design coefficients and factors for seismic force-resisting systems

[Table 12.2-1, page 90-92]

Custom Sail	Structure Type	I. Transverse	II. Longitudinal
C. Moment-R C. Moment-R	Steel moment frames	C. Moment-Resisting Frame Systems	C. Moment-Resisting Frame Systems
4. Steel ordir 4. Steel ordir	3.50 3.50	4. Steel ordinary moment frame	4. Steel ordinary moment frame
- -	3.00 3.00	-	-
- -	3.00 3.00	-	-
	Response modification coefficient	R = 3.50	R = 3.50
	System overstrength factor	$\Omega_0 = 3.00$	$\Omega_0 = 3.00$
	Deflection amplification factor	$C_d = 3.00$	$C_d = 3.00$

Seismic response coefficients

[12.8.1.1, page 101]

Structure seismic load directions	I. Transverse	II. Longitudinal
$C_s = \frac{S_{DS}}{\left(\frac{R}{T}\right)}$	$C_S = 0.039$ Govern!	$C_S = 0.039$ Govern!
For $T \leq T_L$, $C_{s\ max} = \frac{S_{D1}}{T\left(\frac{R}{T}\right)}$	$C_{S\ max.} = 0.230$	$C_{S\ max.} = 0.230$
For $T > T_L$, $C_{s\ max} = \frac{S_{D1}T_L}{T^2\left(\frac{R}{T}\right)}$	$C_{S\ max.} = (n.a.)$	$C_{S\ max.} = (n.a.)$
$C_{s\ min} = 0.044 S_{DS} I_e$	$C_{S\ min.} = 0.006$	$C_{S\ min.} = 0.006$
$C_{s\ min} = 0.01$	$C_{S\ min.} = 0.01$	$C_{S\ min.} = 0.01$
For $S_1 \geq 0.6\ g$, $C_{s\ min} = \frac{0.5S_1}{\left(\frac{R}{T}\right)}$	$C_{S\ min.} = (n.a.)$	$C_{S\ min.} = (n.a.)$
Design seismic response coefficients	$C_S = \mathbf{0.039}$	$C_S = \mathbf{0.039}$

Seismic base shear

$V = C_S W$ [Eq. (12.8-1), page 101]
 $W =$ the effective seismic weight per Section 12.7.2.

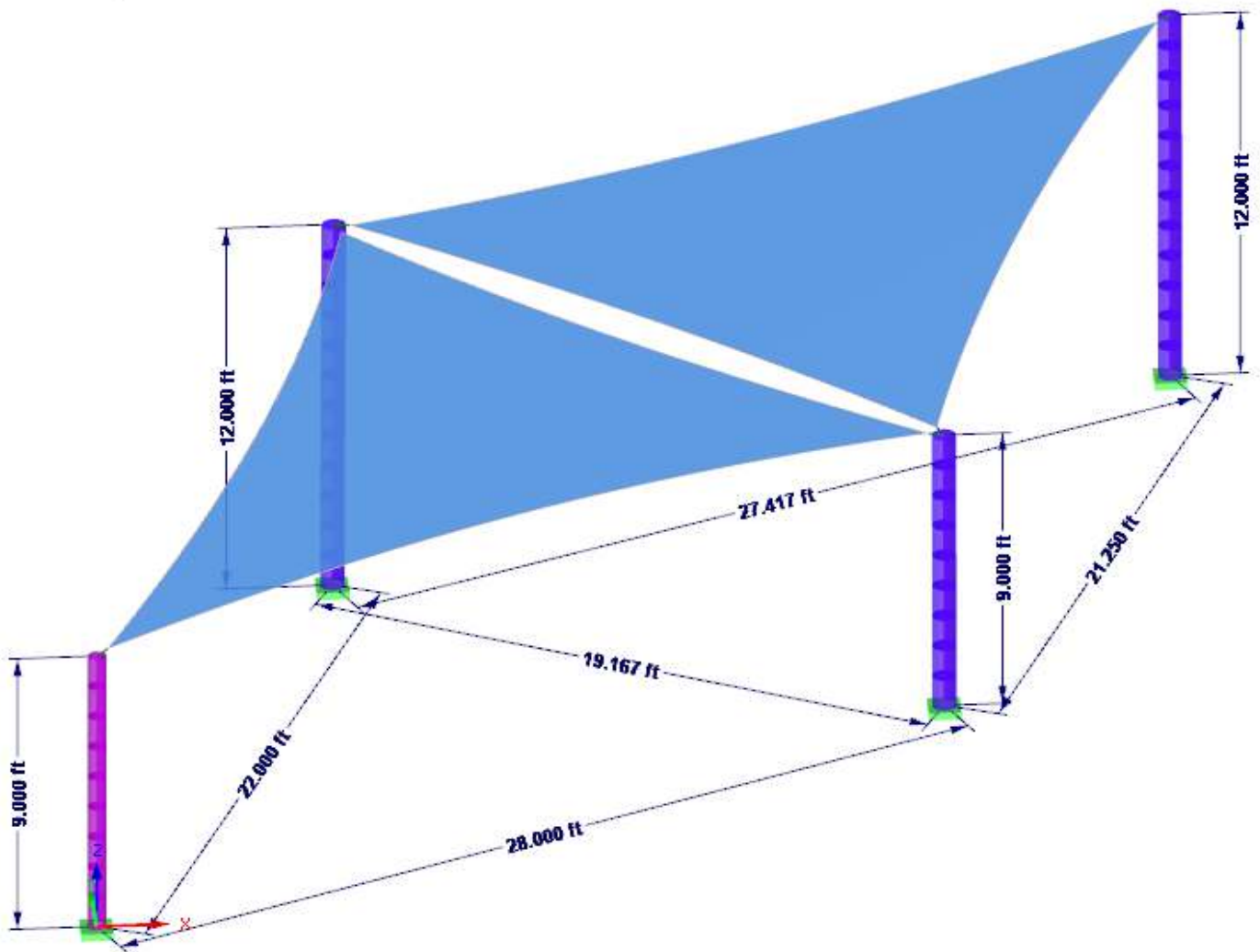
Seismic Force for Component Design

	I. Transverse	II. Longitudinal
For ductile structural elements members & connections	$C_S W_p$ $0.2 S_{DS} W_p$	$F_{p\ horz} = 0.039 W_p$ $F_{p\ vert} = \pm 0.028 W_p$
For nonductile structural elements anchorage to concrete and masonry	$(C_S \Omega_0) W_p$ $0.2 S_{DS} W_p$	$F_{p\ horz} = \mathbf{0.118} W_p$ $F_{p\ vert} = \pm \mathbf{0.028} W_p$

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Finite Element Analysis (FEA)

Geometry and Cross Sections



Cross-Sections

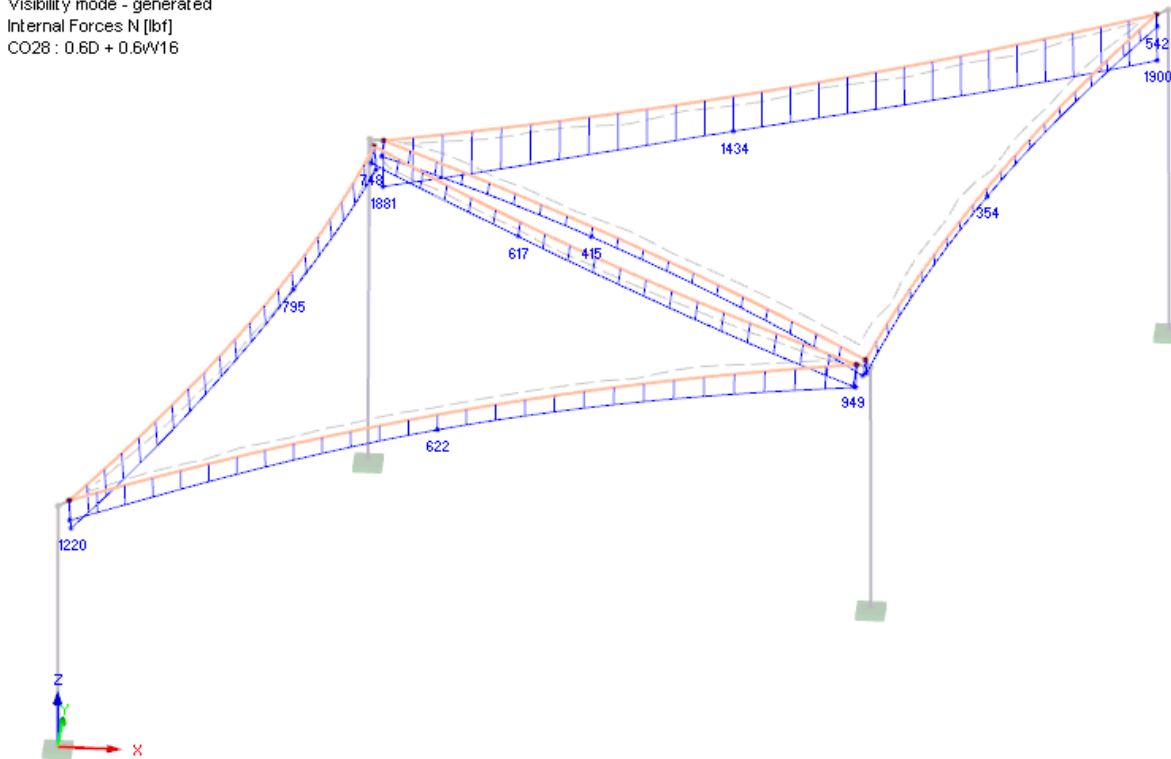
- 12: Pipes Std. 6 | AISC 15; Steel A53, Grade B
- 13: Pipes Std. 8 | AISC 15; Steel A53, Grade B
- 25: Round 0.3125; Cable PG (Pfeifer)
- 64: Round 0.75; Cable PG (Pfeifer)

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Cable Forces

Load Combination **IBC Section 1605.3.1 (ASD)**

Visibility mode - generated
Internal Forces N [lbf]
CO28 : 0.6D + 0.6W16



Max N: 1900, Min N: 354 lbf

[Southern Wire] Vinyl Coated
Strand 7x19 - Stainless T-304/T-316

Cable design strength
[ASCE 19, Section 3.2]

T_a = Allowable tension load (ASD)
 S_n = Breaking strength
 S_d = Cable design strength
 ϕ_f = Fitting reduction factor (1.00)
 $S_d \geq 2.0 T$ 1.00
 $S_d = S_n \times \phi_f$
 $T_a = (S_n \times \phi_f) / 2.0$
 $T \leq T_a$

Cable Size (inches)	Weight lb / ft	Breaking Strength S_n (lb)	Allowable Load T_a (lb)
1/8	0.036	1760	880
3/16	0.079	3700	1850
1/4	0.129	6400	3200
5/16	0.185	9000	4500
3/8	0.270	12000	6000
1/2	0.460	22800	11400
5/8	0.720	35000	17500
3/4	1.040	49600	24800

Threaded rod design strength

Rod Size (inches)	Rod Area in ²	Steel Strength (ksi)	LRFD capacity (lb)	ASD capacity T_a (lb)
3/8	0.110	36	3580	2382
1/2	0.196	36	6364	4234
5/8	0.307	36	9944	6616
3/4	0.442	36	14320	9527
1	0.786	36	25457	16938
1 1/4	1.228	36	39777	26465
1 1/2	1.768	36	57279	38109
1 3/4	2.406	36	77963	51871

Cable design check

Cable Label	Member No.	N, Forces (lb) ASD	Cable size	Allowable Load (lb)	Design Utilization
		1900	5/16	4500	42%

Tension rod design check

Member No.	N, Forces (lb) ASD	Rod size	Allowable Load (lb)	Design Utilization
	2436	3/4	9527	26%

Note: N (+) is tension, N (-) is compression.

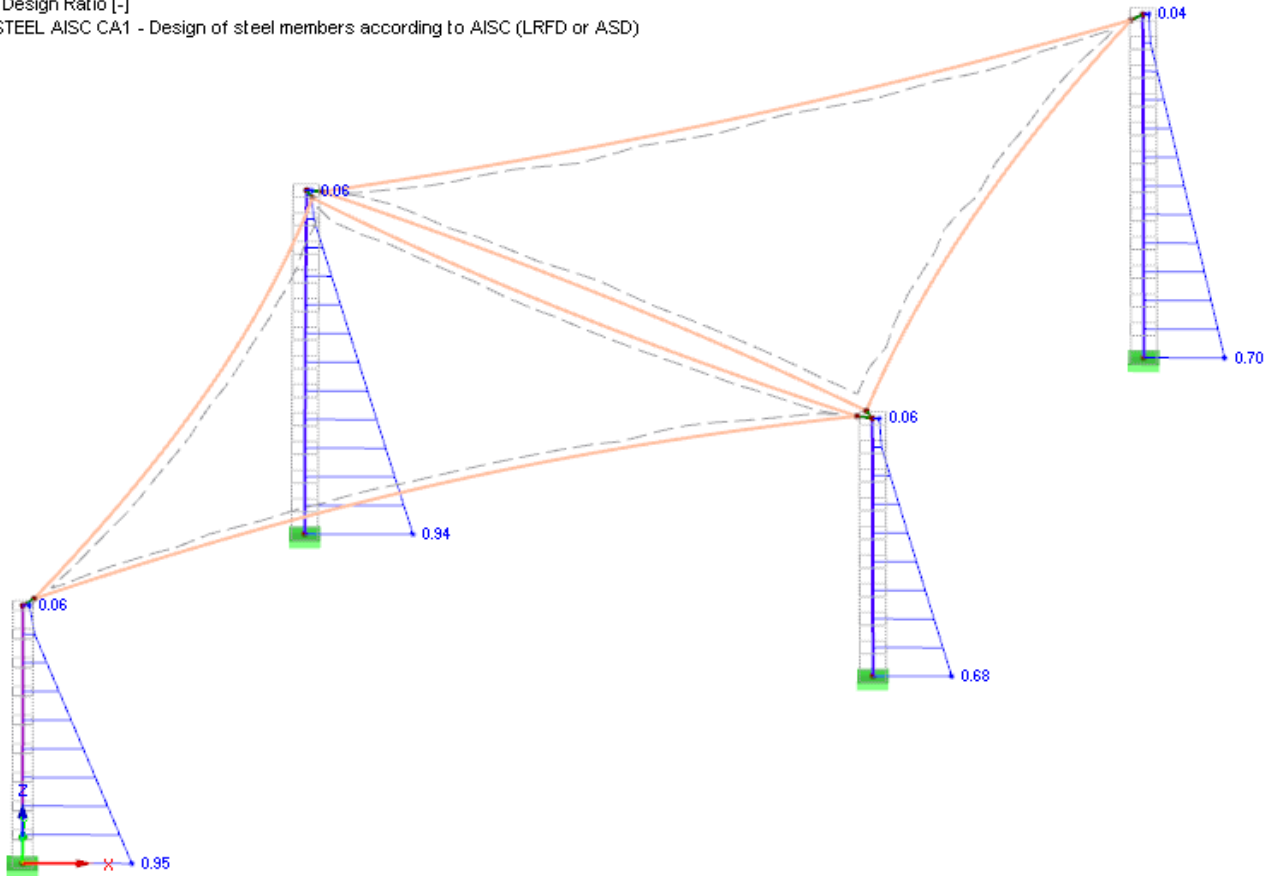
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AISC Steel Check

ID : **Frame with fabric**

Load Combination **IBC Section 1605.2 (LRFD)**

Max Design Ratio [-]
RF-STEEL AISC CA1 - Design of steel members according to AISC (LRFD or ASD)



Max Design Ratio: 0.95

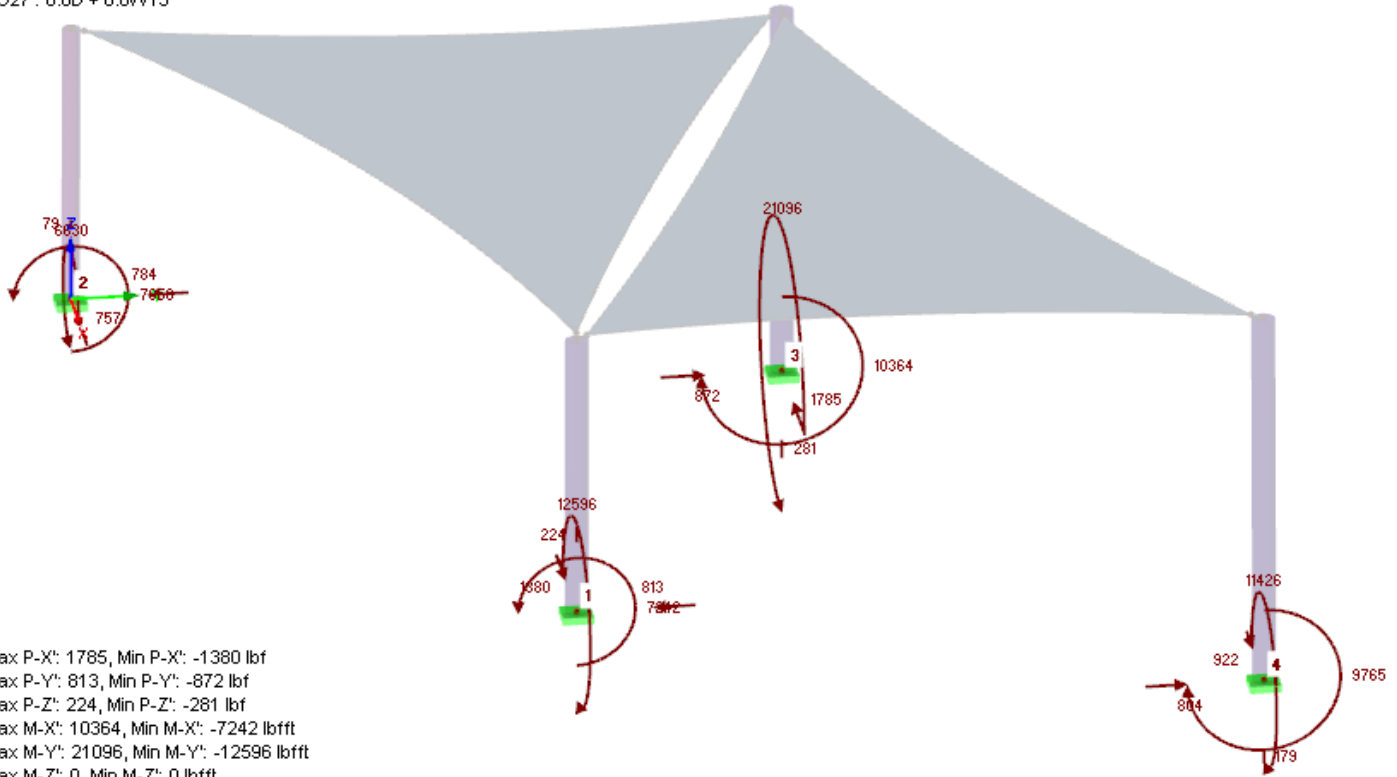
2.1 Design by Load Case

Load- ing	A	B	C	D	E	F	G
	Description	Member No.	Location x [ft]	Design Ratio		Design According to Formula	DS
Ultimate Limit State Design							
CO1	1.4D	3	0.000	0.29	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO2	1.2D + W15	3	0.000	0.52	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO3	1.2D + W16	3	0.000	0.94	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO4	1.2D + W17	3	0.000	0.80	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO5	1.2D + W18	3	0.000	0.41	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO6	1.2D + W19	3	0.000	0.43	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO7	1.2D + W20	1	0.000	0.95	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO8	1.2D + W21	1	0.000	0.85	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO9	1.2D + W22	1	0.000	0.38	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO10	0.9D + W15	3	0.000	0.53	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO11	0.9D + W16	3	0.000	0.94	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO12	0.9D + W17	3	0.000	0.80	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO13	0.9D + W18	3	0.000	0.42	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO14	0.9D + W19	3	0.000	0.42	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO15	0.9D + W20	1	0.000	0.94	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO16	0.9D + W21	1	0.000	0.85	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS
CO17	0.9D + W22	1	0.000	0.37	≤ 1	106) Chapter F - Yielding - Bending about y-axis and/or z-axis acc. to F8 or F11	ULS

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone :	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email :	yurianto@modularconsultant.com	Date :	1/29/2026	Final

Support Reactions R1 ID : Wind at 0 degree

Visibility mode - generated
Support Reactions [lb], [lbfft]
CO27 : 0.6D + 0.6W15



Max P-X: 1785, Min P-X: -1380 lbf
Max P-Y: 813, Min P-Y: -872 lbf
Max P-Z: 224, Min P-Z: -281 lbf
Max M-X: 10364, Min M-X: -7242 lbfft
Max M-Y: 21096, Min M-Y: -12596 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO27 : 0.6D + 0.6W15

CO10 : 0.9D + W15

0.6D + 0.6W

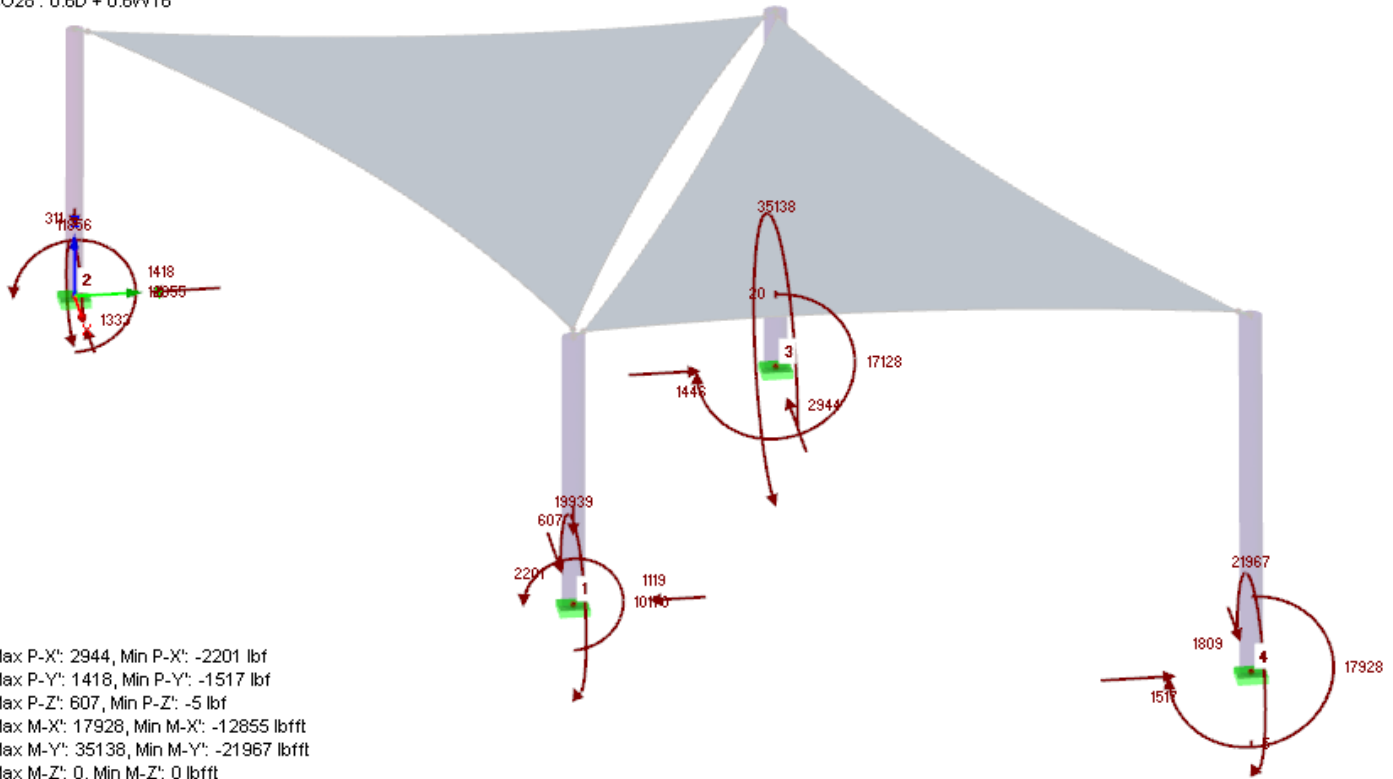
0.9D + W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R1-1	C1	1	-1380	813	224	-7242	-12596	0	-1642	945	269	-8389	-15072	0
R1-2	C2	2	757	784	79	-7058	6630	0	1005	1048	124	-9441	8737	0
R1-3	C3	3	1785	-872	-281	10364	21096	0	2160	-1143	-311	13542	25364	0
R1-4	C4	4	-922	-804	-179	9765	-11426	0	-1123	-984	-233	11998	-14069	0

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Support Reactions R4 ID : Wind at 45 degrees

Visibility mode - generated
Support Reactions [lbf], [lbfft]
CO28 : 0.6D + 0.6W16



Max P-X: 2944, Min P-X: -2201 lbf
Max P-Y: 1418, Min P-Y: -1517 lbf
Max P-Z: 607, Min P-Z: -5 lbf
Max M-X: 17928, Min M-X: -12855 lbfft
Max M-Y: 35138, Min M-Y: -21967 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO28 : 0.6D + 0.6W16
0.6D + 0.6W

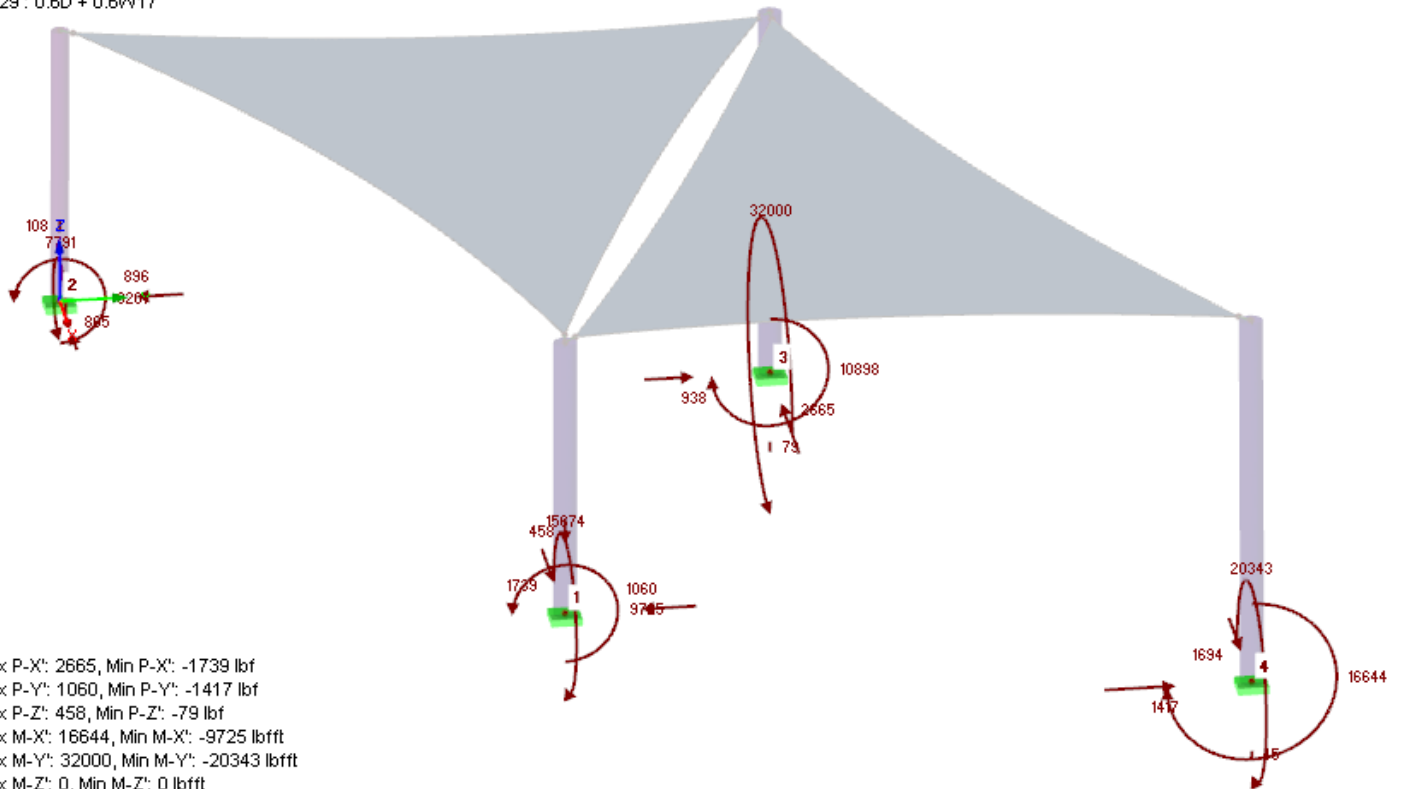
CO11 : 0.9D + W16
0.9D + W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R4-1	C1	1	-2201	1119	607	-10170	-19939	0	-2786	1378	875	-12559	-25282	0
R4-2	C2	2	1333	1418	311	-12855	11856	0	1835	1963	495	-17810	16277	0
R4-3	C3	3	2944	-1446	20	17128	35138	0	3822	-1988	233	23465	45503	0
R4-4	C4	4	-1809	-1517	-5	17928	-21967	0	-2426	-2062	62	24279	-29541	0

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Support Reactions R5 ID : Wind at 90 degrees

Visibility mode - generated
Support Reactions [lbf], [lbfft]
CO29 : 0.6D + 0.6W17



Max P-X: 2665, Min P-X: -1739 lbf
Max P-Y: 1060, Min P-Y: -1417 lbf
Max P-Z: 458, Min P-Z: -79 lbf
Max M-X: 16644, Min M-X: -9725 lbfft
Max M-Y: 32000, Min M-Y: -20343 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO29 : 0.6D + 0.6W17
0.6D + 0.6W

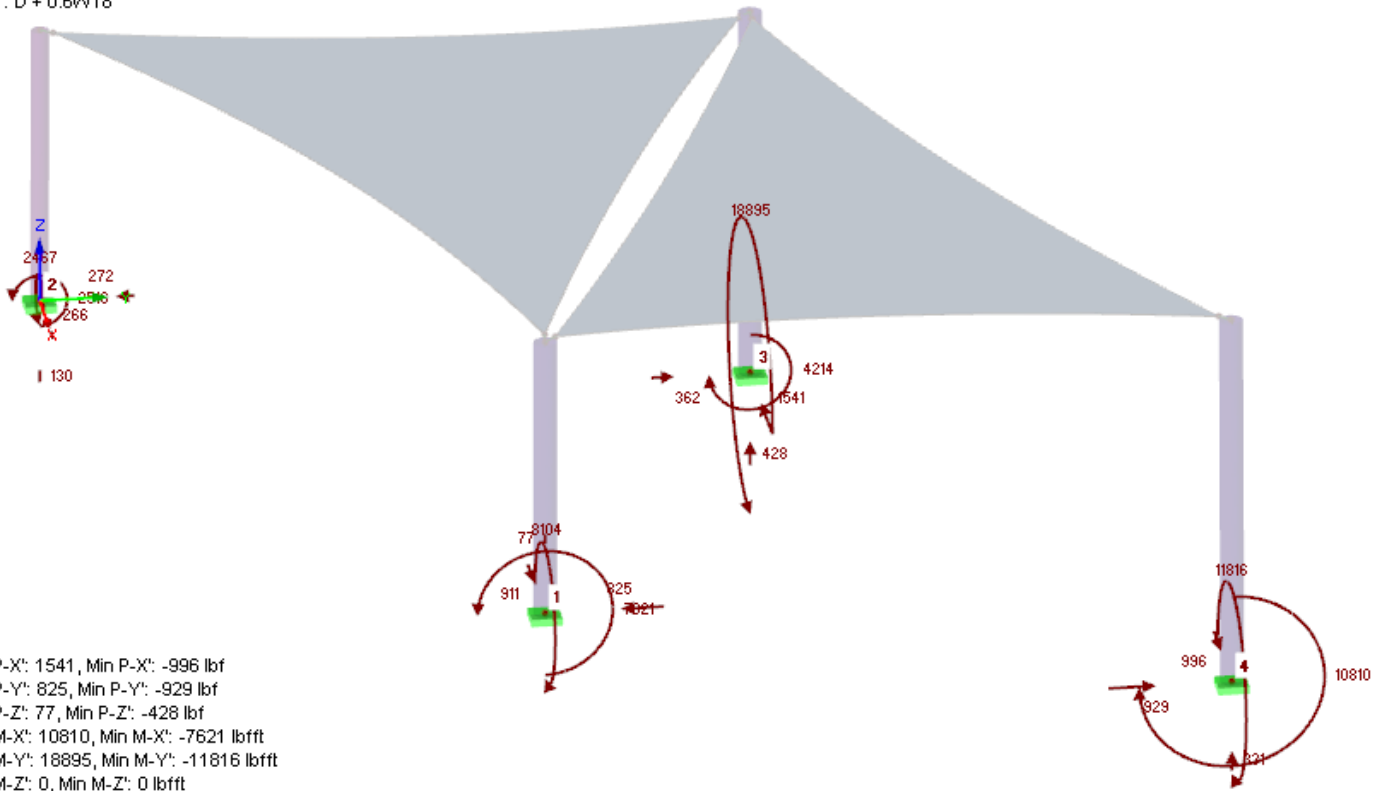
CO12 : 0.9D + W17
0.9D + W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R5-1	C1	1	-1739	1060	458	-9725	-15674	0	-2169	1285	641	-11873	-19562	0
R5-2	C2	2	865	896	108	-8201	7791	0	1162	1214	170	-11156	10458	0
R5-3	C3	3	2665	-938	-79	10898	32000	0	3433	-1260	39	14521	41202	0
R5-4	C4	4	-1694	-1417	-45	16644	-20343	0	-2262	-1903	-4	22240	-27170	0

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Support Reactions R6 ID : Wind at 135 degrees

Visibility mode - generated
Support Reactions [lbf], [lbfft]
CO22 : D + 0.6W18



Max P-X: 1541, Min P-X: -996 lbf
Max P-Y: 825, Min P-Y: -929 lbf
Max P-Z: 77, Min P-Z: -428 lbf
Max M-X: 10810, Min M-X: -7621 lbfft
Max M-Y: 18895, Min M-Y: -11816 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO22 : D + 0.6W18

CO5 : 1.2D + W18

D + 0.6W

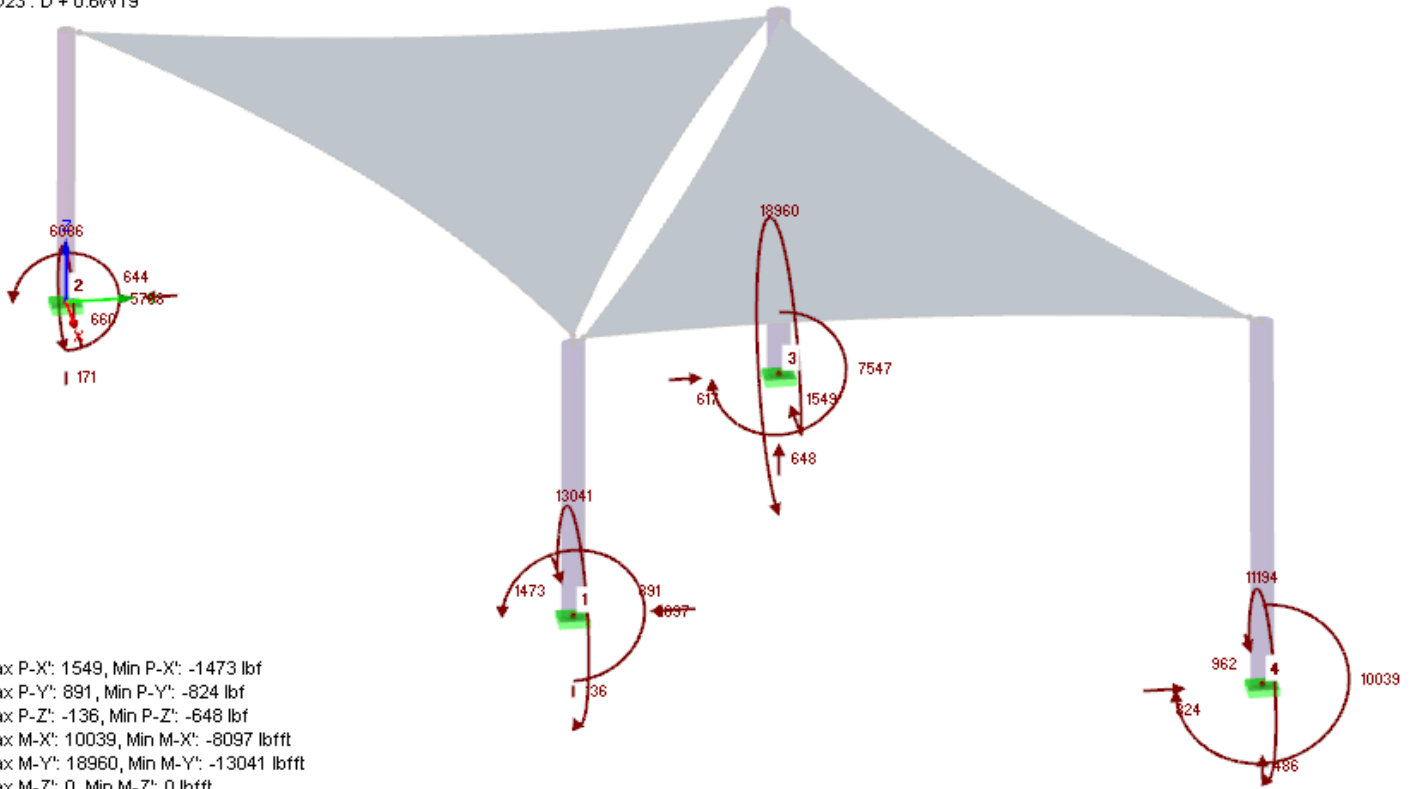
1.2D+W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R6-1	C1	1	-911	825	77	-7621	-8104	0	-1005	976	133	-9111	-8893	0
R6-2	C2	2	266	272	-130	-2516	2467	0	273	276	-155	-2601	2577	0
R6-3	C3	3	1541	-362	-428	4214	18895	0	1795	-391	-451	4476	22203	0
R6-4	C4	4	-996	-929	-331	10810	-11816	0	-1228	-1185	-365	13647	-14518	0

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Support Reactions R7 ID : **Wind at 180 degrees**

Visibility mode - generated
 Support Reactions [lb], [lbft]
 CO23 : D + 0.6W19



Max P-X: 1549, Min P-X: -1473 lbf
 Max P-Y: 891, Min P-Y: -824 lbf
 Max P-Z: -136, Min P-Z: -648 lbf
 Max M-X: 10039, Min M-X: -8097 lbft
 Max M-Y: 18960, Min M-Y: -13041 lbft
 Max M-Z: 0, Min M-Z: 0 lbft

CO23 : D + 0.6W19

CO6 : 1.2D + W19

D + 0.6W

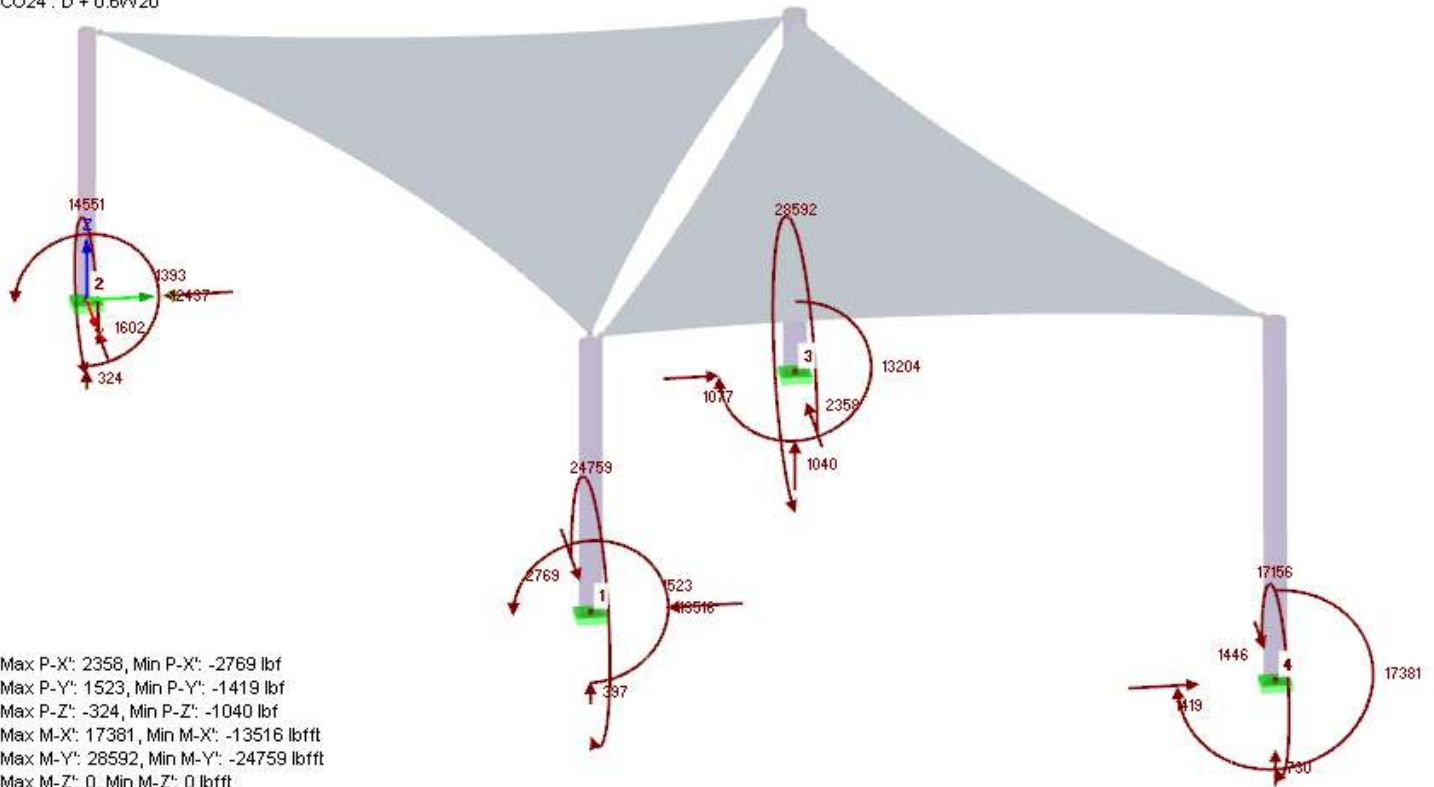
1.2D+W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R7-1	C1	1	-1473	891	-136	-8097	-13041	0	-1816	1080	-233	-9847	-15986	0
R7-2	C2	2	660	644	-171	-5788	6086	0	853	814	-232	-7320	7917	0
R7-3	C3	3	1549	-617	-648	7547	18960	0	1745	-730	-798	8999	21553	0
R7-4	C4	4	-962	-824	-486	10039	-11194	0	-1158	-1007	-624	12341	-13316	0

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Support Reactions R8 ID : Wind at 225 degrees

Visibility mode - generated
Support Reactions [lbf], [lbfft]
CO24 : D + 0.6W20



Max P-X: 2358, Min P-X: -2769 lbf
Max P-Y: 1523, Min P-Y: -1419 lbf
Max P-Z: -324, Min P-Z: -1040 lbf
Max M-X: 17381, Min M-X: -13516 lbfft
Max M-Y: 28592, Min M-Y: -24759 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO24 : D + 0.6W20

CO7 : 1.2D + W20

D + 0.6W

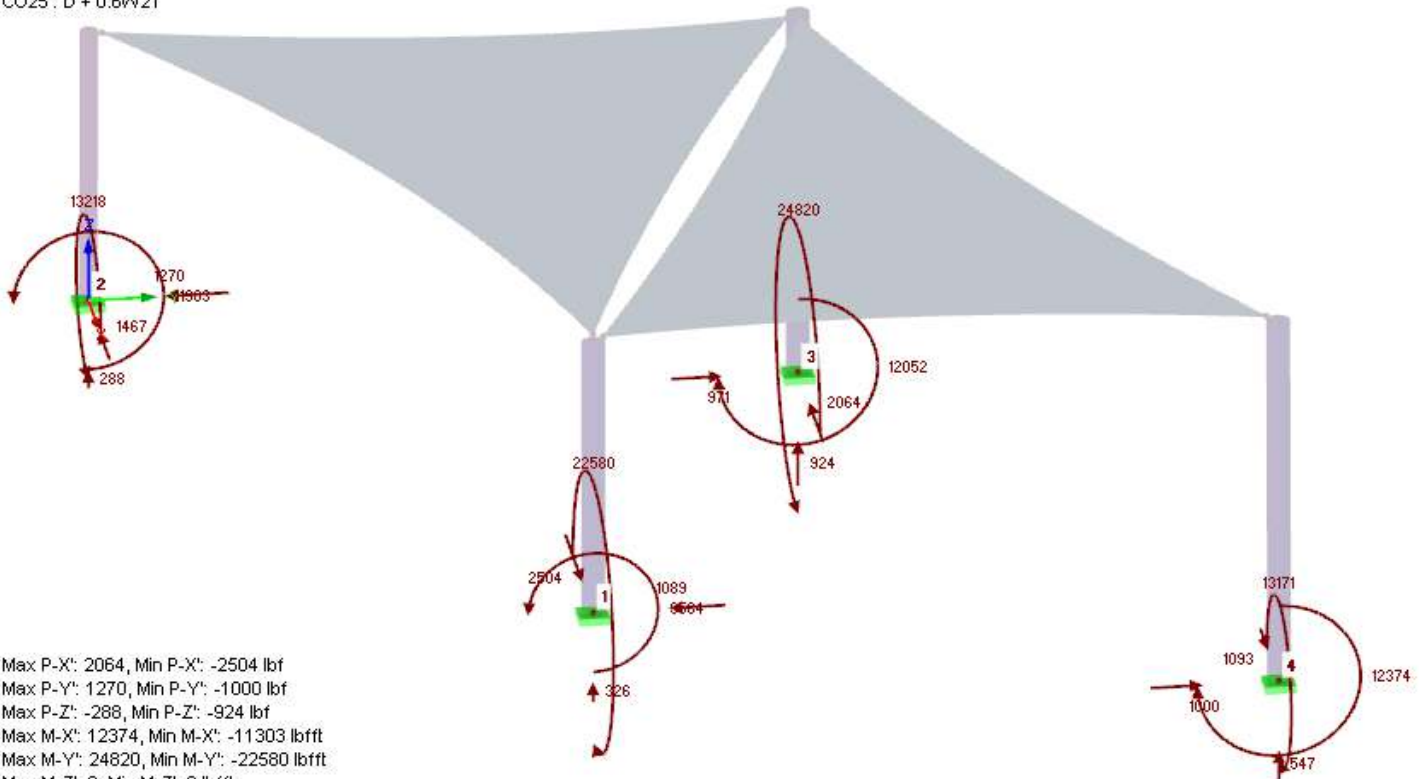
1.2D+W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R8-1	C1	1	-2769	1523	-397	-13516	-24759	0	-3646	2055	-711	-18187	-32555	0
R8-2	C2	2	1602	1393	-324	-12437	14551	0	2212	1905	-497	-16988	20149	0
R8-3	C3	3	2358	-1077	-1040	13204	28592	0	2871	-1362	-1407	16820	34958	0
R8-4	C4	4	-1446	-1419	-730	17381	-17156	0	-1864	-1898	-1020	23375	-22051	0

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Support Reactions R9 ID : **Wind at 270 degrees**

Visibility mode - generated
Support Reactions [lbf], [lbfft]
CO25 : D + 0.6W21



Max P-X: 2064, Min P-X: -2504 lbf
Max P-Y: 1270, Min P-Y: -1000 lbf
Max P-Z: -288, Min P-Z: -924 lbf
Max M-X: 12374, Min M-X: -11303 lbfft
Max M-Y: 24820, Min M-Y: -22580 lbfft
Max M-Z: 0, Min M-Z: 0 lbfft

CO25 : D + 0.6W21

CO8 : 1.2D + W21

D + 0.6W

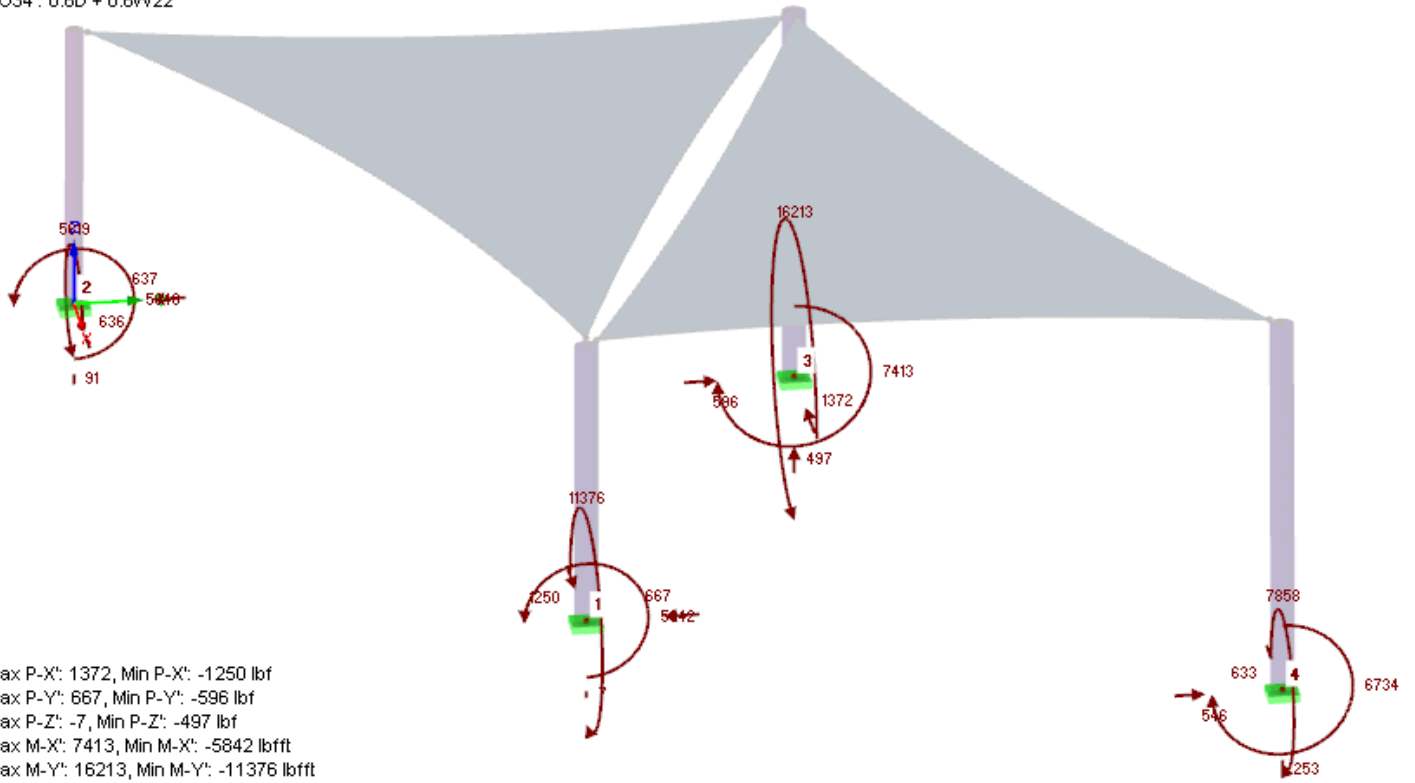
1.2D+W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R9-1	C1	1	-2504	1089	-326	-9564	-22580	0	-3283	1414	-571	-12339	-29630	0
R9-2	C2	2	1467	1270	-288	-11303	13218	0	2022	1722	-437	-15289	18223	0
R9-3	C3	3	2064	-971	-924	12052	24820	0	2500	-1210	-1229	15180	30078	0
R9-4	C4	4	-1093	-1000	-547	12374	-13171	0	-1348	-1279	-725	15986	-16263	0

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Support Reactions R10 ID : **Wind at 315 degrees**

Visibility mode - generated
 Support Reactions [lb], [lbfft]
 CO34 : 0.6D + 0.6W22



Max P-X: 1372, Min P-X: -1250 lbf
 Max P-Y: 667, Min P-Y: -596 lbf
 Max P-Z: -7, Min P-Z: -497 lbf
 Max M-X: 7413, Min M-X: -5842 lbfft
 Max M-Y: 16213, Min M-Y: -11376 lbfft
 Max M-Z: 0, Min M-Z: 0 lbfft

CO34 : 0.6D + 0.6W22
 0.6D + 0.6W

CO17 : 0.9D + W22
 0.9D + W

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb)			Support Moment (lb-ft)			Support Forces (lb)			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R10-1	C1	1	-1250	667	-7	-5842	-11376	0	-1497	718	-107	-6188	-13675	0
R10-2	C2	2	636	637	-91	-5648	5619	0	833	828	-159	-7307	7326	0
R10-3	C3	3	1372	-596	-497	7413	16213	0	1502	-723	-680	9110	17610	0
R10-4	C4	4	-633	-546	-253	6734	-7858	0	-631	-551	-356	6925	-8017	0

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		Date:	1/29/2026	Final

FOUNDATION

[IBC, Chapter 18]

Presumptive load-bearing values

[Table 1806.2]

No.	Class of material	Vertical foundation pressure (psf)	Lateral bearing pressure (below natural grade) (psf / ft)	Lateral sliding resistance	
				Coefficient of friction ^a	Cohesion ^b (psf)
1	Crystalline bedrock	12000	1200	0.7	
2	Sedimentary and foliated rock	4000	400	0.35	
3	Sandy gravel and/or gravel (GW and GP)	3000	200	0.35	
4	Sand, silty sand, clayey sand silty gravel and clayey gravel (SW, SP, SM, SC, GM & GC)	2000	150	0.25	
5	Clay, sandy clay, silty clay clayey silt, silt and sandy silt (CL, ML, MH and CH)	1500	100		130

a. Coefficient to be multiplied by the dead load

b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2

Soil Class of material		5	Clay, sandy clay, silty clay
Vertical foundation pressure	1500	1500 psf	
Lateral bearing pressure	100	100 psf/ft	(below natural grade)
Coefficient of friction	0	0	
Cohesion	130	130 psf	

Design Situation Options

Design Situations	(Adj. factor)	Code Provisions
Applicable	2.00	Isolated poles for uses such as flagpoles and poles shall be permitted to be designed using lateral bearing pressures equal to two times the tabulated values, according to 1806.3.4.
Not Applicable	1.00	The values of vertical foundation pressure and lateral bearing pressure given in Table 1806.2 are increased by one-third, according to 1806.1, where used with the alternative basic load combinations of Section 1605.3.2 that include wind or earthquake loads.
Not Applied		Alternative basic load combination per IBC, Section 1605.3.2. (ASD)
Not Applied		Counteracting effects of dead load and wind loads, only two-thirds of dead load are used. Also, coefficient ω in Equations 16-18, 16-19 and 16-20 shall be increased to 1.3 (otherwise $\omega = 1.0$), according to 1605.3.2. Example: 1.0D+0.6W becomes 0.67D + 0.78W.

Design load-bearing values

Presumptive load-bearing values times applicable adjustment factors of design situations.

Vertical foundation pressure	1500	1500 psf	
Lateral bearing pressure	200	200 psf / ft	
Allowable friction resistance (AFR)	250	250 psf	1/6 vertical pressure, or 500 psf max. [1810.3.3.1.4]

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Selected Support Reaction Governing Case R8 **Wind at 225 degrees** **RWIND**

Column Label **C1**

Node No. **1**

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R8-1	C1	1	-2769	1523	-397	-13516	-24759	0	-3646	2055	-711	-18187	-32555	0

Drilled Piers ID : **C1**

Load combination **IBC Section 1605.3.1 (ASD)**

Trial and Error - Counterweight	10932		10931 lb-ft			[Calcs next page]
Applied Moment	17277	M =	17277 lb-ft	M = $\sqrt{(M_x^2 + M_y^2)}$ - Counterweight		
Applied Lateral force	3160	P =	3160 lb	$P = \sqrt{(P_x^2 + P_y^2)}$		
Vertical downward	397	V _{down} =	397 lb	Utilization	47%	
Vertical uplift	0	V _{uplift} =	0 lb	Utilization	0%	

Determine pier embedment to resist lateral loads

Footing shape **Round**
Round diameter **36.0** inches

Diameter b = 3.00 ft [36.0"] 3'-0" [1810.2.2]
Distance from ground to point of application "P" 5.47 h = 5.47 ft (Moment / Lateral force)

Ground surface condition **Nonconstrained**

Trial and Error - Depth of embedment d_{trial} = **8.03** ft
Δ = 0.00 ft

Minimum embedment depth d_{min} = 8.03 ft [96.3"] 8'-0 1/4 "

Use depth of embedment **d = 8.08 ft** [97.0"] 8'-1"

Footing total height 8.08 L = **8.08** ft [97.0"] 8'-1" [1810.2.2]

Minimum pier embedment depth calculations [1807.3.2]

Nonconstrained d = 0.5A { 1 + [1 + (4.36 h/A)]^{1/2} } where, A = 2.34 P / (S₁ b) [1807.3.2.1]
S₁ = 535 psf lateral bearing pressure x embedment depth (max. 12ft) / 3
A = 4.61
d_{min} = 8.03 ft [96.3"] 8'-0 1/4 "

Constrained d = $\sqrt{4.25 P h / (S_3 b)}$ [1807.3.2.2]
S₃ = 1605 psf lateral bearing pressure x embedment depth (max. 12ft)
d_{min} = 3.90 ft [46.9"] 3'-10 3/4 "

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Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Pier load capacity

Round	b =	3.00	ft	[36.0"]	3'-0"	
Pier perimeter		9.43	ft	[113.1"]		
Pier bearing area		7.07	ft ²	1018	in ²	
Effective skin area		76.21	ft ²	Embedment, excluding upper	0.0	feet
Pier volume		57.16	ft ³	2.117	cubic yard	
Gross pier weight		8574	lb	Unit weight of concrete	150	lb/ft ³
Net pier weight for compressive load		8574	lb	Conc. - Soil weight	0	150 lb/ft ³
Net pier weight for tensile load		4973	lb	Bouyant weight of concrete	87	lb/ft ³
Counterweight		10932	lb-ft	Pier weight x moment arm, which (0.85 x 1/2 pier diameter)		
Total bearing load		8971	lb	Vertical downward + pier weight		
Bearing resistance		10607	lb	Bearing area x bearing pressure capacity		
Skin friction resistance		19054	lb	AFR x effective skin area		
Allowable downward resistance		19054	lb	Maximum of bearing resistance or skin friction resistance		
Utilization		47%	ok!			
Allowable uplift resistance		24027	lb	(Pier weight + skin friction resistance)		
Utilization		0%	ok!	(No uplift)		

Pier concrete reinforcements

Design Criteria

Soil Site Class	D	D	
Deep foundation type		Cast-in-place	[1810.3.9]
Seismic Design Category	2	B	B
Min. Concrete Strength	2500	f _c =	3000 psi [1808.8.1]
Reinforced Rebar Yield Strength		f _y =	60 ksi

Longitudinal bars

Bar size		#6 (3/4")	[1810.3.9.4.1]
Bar diameter	d _b =	0.750	in
Bar area	A _{s1} =	0.442	in ²
Number of bars	12	12	bar(s)
Total bar area	A _s =	5.30	in ² ok!
Min. reinforcement	0.0025	0.005	pier area
	A _{s min} =	5.09	in ²
Concrete cover		2.50	in [1808.8.2]
Rebar spacing	s =	8.12	in ok!
Min. spacing		1.50	in [ACI-318, 7.6.3.]

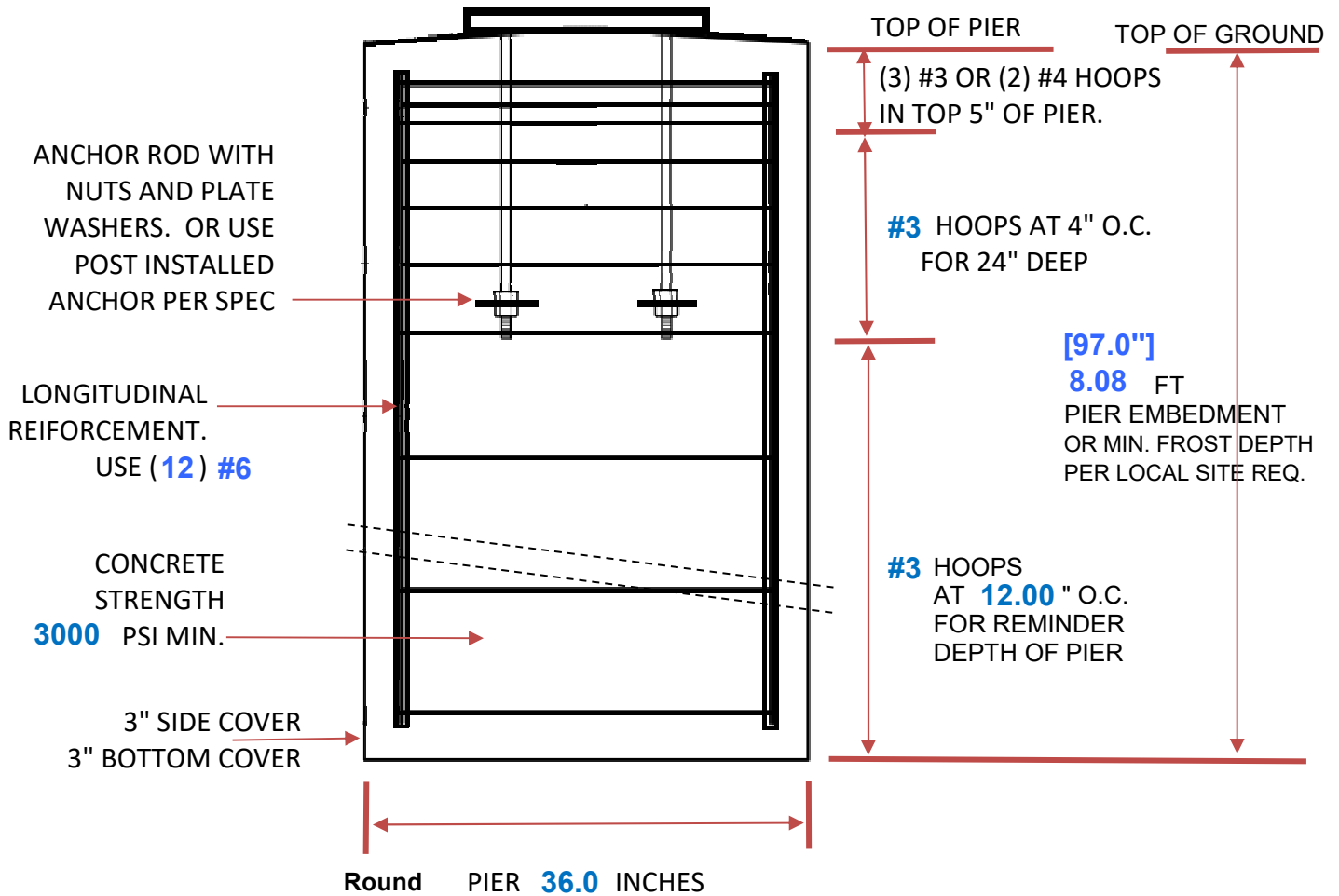
Transverse Ties

Bar size	#3 (3/8")	#3 (3/8")	#3 (3/8") #4 (1/2")
Bar diameter		d _b =	0.375 in
Top Ties	(3) #3 OR (2) #4	hoops in top 5" of Pier	
Tie spacing witin 24" deep from top ties		4.00	in o.c.
Reminding Ties Minimum Spacing		12.00	in

12.00

Engineer :	Yurianto Yuriyanto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Drilled Pier Sketch

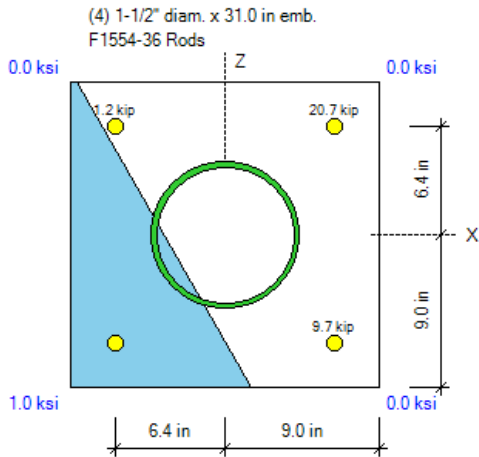


Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Base Plate & Anchor

ID : C1

Load Combination **IBC Section 1605.2 (LRFD)**



PLAN

Design Ratios	
Bearing	0.30 ✓
Anchorage	0.59 ✓

GEOMETRY

	Width	Length
Pipe8STD	8.6	8.6
Steel Column	8.6	8.6
Steel Plate	18.0	18.0
Concrete Support	18.0	18.0
Rod Edge	2.63	2.63
Thickness of Grout	1.0	1.0

DIMENSIONS ARE OK

FACTORED LOADS (LRFD)

Vertical Load P	0.7	kip		
Moment Mx	-18.2	Mz	-32.6	k-ft
Shear Vx	-3.7	Vz	2.1	kip
Eccentr. ex	558.9	ez	312.0	in

BEARING STRESSES

Concrete Strength f'c	3.0	ksi
Minimum Stress	0.0	ksi
Maximum Stress fp	1.0	ksi
Bearing Strength ϕF_{pn}	3.3	ksi
$f_p / \phi F_{pn}$ Bearing Design Ratio	0.30	✓

BEARING STRESSES ARE OK

BASE PLATE DESIGN

Steel Strength Fy	36.0	ksi
<i>Plate Allowed to Work in the Plastic Range</i>		
Maximum Plate Moment	10.4	k-in/in
Thickness Required	1.13	in
Use Plate Thickness	1.50	in ✓

ANCHORAGE DESIGN

Anchor Rod Material	F1554-36	
(4) Rods , fya = 36.0 ksi, futa = 58.0 ksi		
Rod Size	1-1/2" diam. x 31.0 in emb.	
<u>-Tension</u>		
Total Tension Force N	31.6	kip
Max. Tension per Rod	20.7	kip
$N / \phi N_n$ Tension Design Ratio	0.57	✓
<u>-Shear</u>		
Shear Taken by	Anchor Rods only	
Total Shear Force V	4.3	kip
Shear Force per Rod Vi	2.1	kip
<i>Only Front Rods Are Effective</i>		
<i>Anchor Reinforcement Provided</i>		
$V / \phi V_n$ Shear Design Ratio	0.10	✓
<u>-Tension-Shear Interaction</u>		
Combined Stress Ratio	0.59	✓

ANCHORAGE DESIGN IS OK

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Selected Support Reaction Governing Case R8 **Wind at 225 degrees** **RWIND**

Column Label **C2**

Node No. **2**

Reaction	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
Line ID			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R8-2	C2	2	1602	1393	-324	-12437	14551	0	2212	1905	-497	-16988	20149	0

Drilled Piers ID : **C2**

Load combination **IBC Section 1605.3.1 (ASD)**

Trial and Error - Counterweight	9016		9015 lb-ft			[Calcs next page]
Applied Moment	10127	M =	10127 lb-ft	M = $\sqrt{(M_x^2 + M_y^2)}$ - Counterweight		
Applied Lateral force	2123	P =	2123 lb	$P = \sqrt{(P_x^2 + P_y^2)}$		
Vertical downward	324	V _{down} =	324 lb	Utilization 47%		
Vertical uplift	0	V _{uplift} =	0 lb	Utilization 0%		

Determine pier embedment to resist lateral loads

Footing shape **Round**
 Round diameter **36.0** inches

Diameter	b =	3.00 ft	[36.0"]	3'-0"	[1810.2.2]
Distance from ground to point of application "P"	h =	4.77 ft	(Moment / Lateral force)		

Ground surface condition **Nonconstrained**

Trial and Error - Depth of embedment	d _{trial} =	6.65 ft			
Δ = 0.00 ft					
Minimum embedment depth	d _{min} =	6.65 ft	[79.8"]	6'-7 3/4 "	
Use depth of embedment	d =	6.67 ft	[80.0"]	6'-8"	
Footing total height	L =	6.67 ft	[80.0"]	6'-8"	[1810.2.2]

Minimum pier embedment depth calculations [1807.3.2]

Nonconstrained	d =	$0.5A \{1 + [1 + (4.36 h/A)]^{1/2}\}$	where, A = 2.34 P / (S ₁ b)	[1807.3.2.1]
	S ₁ =	443 psf	lateral bearing pressure x embedment depth (max. 12ft) / 3	
	A =	3.73		
	d _{min} =	6.65 ft	[79.8"]	6'-7 3/4 "
Constrained	d =	$\sqrt{4.25 P h / (S_3 b)}$		[1807.3.2.2]
	S ₃ =	1330 psf	lateral bearing pressure x embedment depth (max. 12ft)	
	d _{min} =	3.28 ft	[39.4"]	3'-3 3/8 "

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Pier load capacity

Round	b =	3.00	ft	[36.0"]	3'-0"	
Pier perimeter		9.43	ft	[113.1"]		
Pier bearing area		7.07	ft ²	1018	in ²	
Effective skin area		62.86	ft ²	Embedment, excluding upper	0.0	feet
Pier volume		47.14	ft ³	1.746	cubic yard	
Gross pier weight		7071	lb	Unit weight of concrete	150	lb/ft ³
Net pier weight for compressive load		7071	lb	Conc. - Soil weight	0	150 lb/ft ³
Net pier weight for tensile load		4101	lb	Bouyant weight of concrete	87	lb/ft ³
Counterweight		9016	lb-ft	Pier weight x moment arm, which (0.85 x 1/2 pier diameter)		
Total bearing load		7395	lb	Vertical downward + pier weight		
Bearing resistance		10607	lb	Bearing area x bearing pressure capacity		
Skin friction resistance		15714	lb	AFR x effective skin area		
Allowable downward resistance		15714	lb	Maximum of bearing resistance or skin friction resistance		
Utilization		47%	ok!			
Allowable uplift resistance		19816	lb	(Pier weight + skin friction resistance)		
Utilization		0%	ok!	(No uplift)		

Pier concrete reinforcements

Design Criteria

Soil Site Class	D	D	
Deep foundation type		Cast-in-place	[1810.3.9]
Seismic Design Category	2	B	B
Min. Concrete Strength	2500	f _c =	3000 psi [1808.8.1]
Reinforced Rebar Yield Strength		f _y =	60 ksi

Longitudinal bars

Bar size		#6 (3/4")	[1810.3.9.4.1]
Bar diameter	d _b =	0.750	in
Bar area	A _{s1} =	0.442	in ²
Number of bars	12	12	bar(s)
Total bar area	A _s =	5.30	in ² ok!
Min. reinforcement	0.0025	0.005	pier area
		A _{s min} =	5.09 in ²
Concrete cover		2.50	in [1808.8.2]
Rebar spacing	s =	8.12	in ok!
Min. spacing		1.50	in [ACI-318, 7.6.3.]

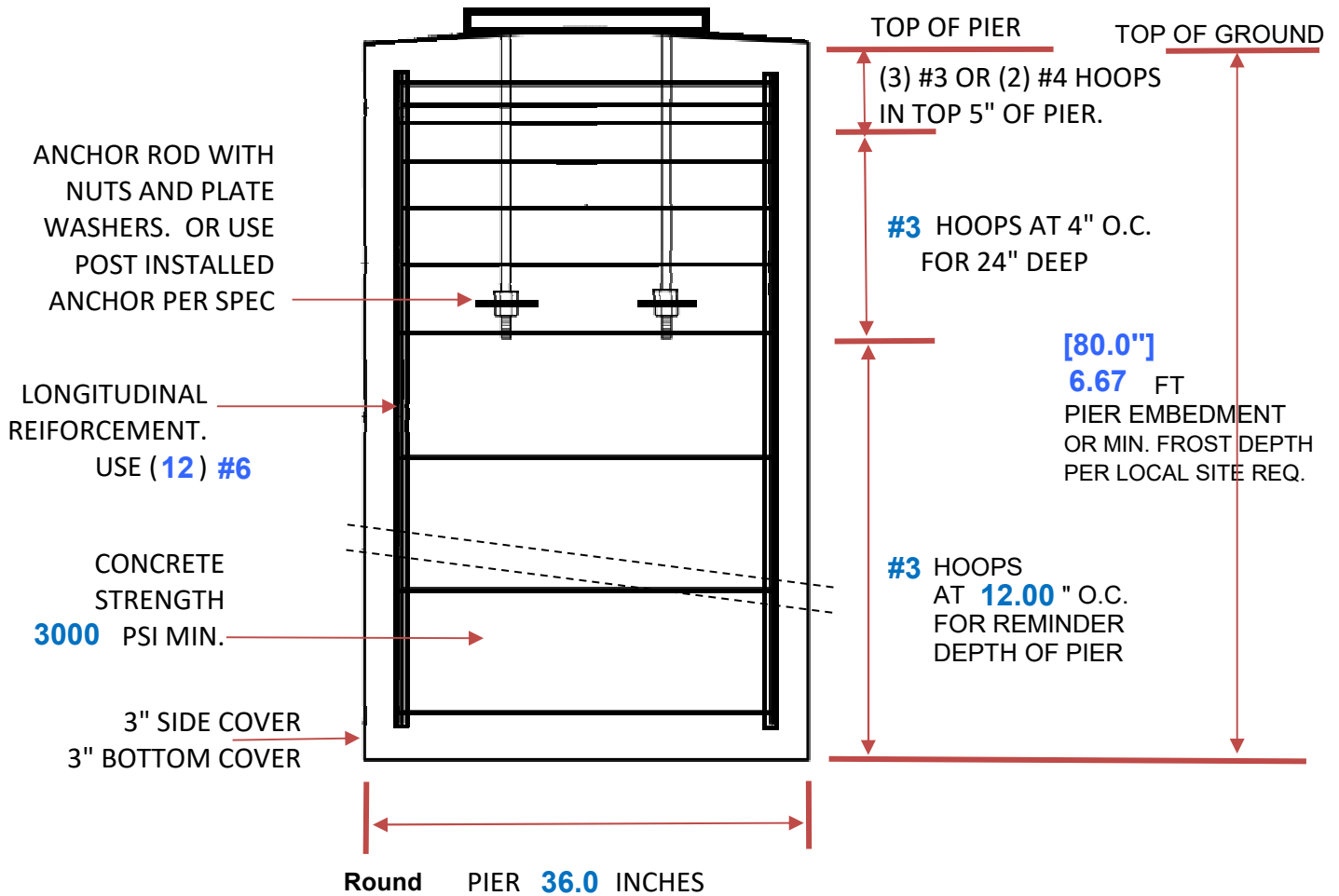
Transverse Ties

Bar size	#3 (3/8")	#3 (3/8")	#3 (3/8") #4 (1/2")	[1810.3.9.4.1]
Bar diameter		d _b =	0.375	in
Top Ties	(3) #3 OR (2) #4	hoops in top 5" of Pier		
Tie spacing witin 24" deep from top ties		4.00	in o.c.	
Reminding Ties Minimum Spacing		12.00	in	

12.00

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Drilled Pier Sketch

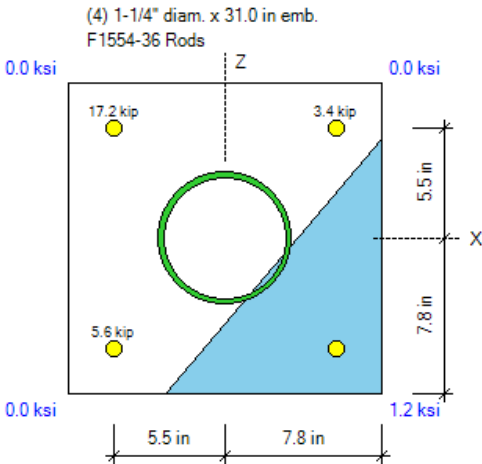


Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Base Plate & Anchor

ID : C2

Load Combination **IBC Section 1605.2 (LRFD)**



PLAN

Design Ratios	
Bearing	0.36 ✓
Anchorage	0.48 ✓

GEOMETRY

	Width	Length
Pipe6STD		
Steel Column	6.6	6.6 in
Steel Plate	15.5	15.5 in
Concrete Support	Wp1 18.0	Lp1 18.0 in
	Wp2 18.0	Lp2 18.0 in
Rod Edge	2.25	2.25 in
Thickness of Grout	1.0	in

DIMENSIONS ARE OK

FACTORED LOADS (LRFD)

Vertical Load P	0.5	kip
Moment Mx	-17.0	Mz ... 20.2 k-ft
Shear Vx	2.2	Vz ... 2.0 kip
Eccentr. ex	484.8	ez ... 408.0 in

BEARING STRESSES

Concrete Strength f'c	3.0	ksi
Minimum Stress	0.0	ksi
Maximum Stress fp	1.2	ksi
Bearing Strength ϕF_{pn}	3.3	ksi
fp / ϕF_{pn} Bearing Design Ratio	0.36	✓

BEARING STRESSES ARE OK

BASE PLATE DESIGN

Steel Strength Fy	36.0	ksi
<i>Plate Allowed to Work in the Plastic Range</i>		
Maximum Plate Moment	8.6	k-in/in
Thickness Required	1.03	in
Use Plate Thickness	1.50	in ✓

ANCHORAGE DESIGN

Anchor Rod Material	F1554-36
(4) Rods , fya = 36.0 ksi, futa = 58.0 ksi	
Rod Size	1-1/4" diam. x 31.0 in emb.
- Tension	
Total Tension Force N	26.2 kip
Max. Tension per Rod	17.2 kip
N / ϕN_n Tension Design Ratio	0.47 ✓
- Shear	
Shear Taken by	Anchor Rods only
Total Shear Force V	3.0 kip
Shear Force per Rod Vi	1.5 kip
<i>Only Front Rods Are Effective</i>	
<i>Anchor Reinforcement Provided</i>	
V / ϕV_n Shear Design Ratio	0.08 ✓
- Tension-Shear Interaction	
Combined Stress Ratio	0.48 ✓

ANCHORAGE DESIGN IS OK

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Selected Support Reaction Governing Case R4 **Wind at 45 degrees** **RWIND**

Column Label **C3**

Node No. **3**

Reaction	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
Line ID			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R4-3	C3	3	2944	-1446	20	17128	35138	0	3822	-1988	233	23465	45503	0

Drilled Piers ID : **C3**

Load combination **IBC Section 1605.3.1 (ASD)**

Trial and Error - Counterweight	11946		11945 lb-ft		[Calcs next page]
Applied Moment	27145	M =	27145 lb-ft	M = $\sqrt{(M_x^2 + M_y^2)}$ - Counterweight	
Applied Lateral force	3280	P =	3280 lb	P = $\sqrt{(P_x^2 + P_y^2)}$	
Vertical downward	0	V _{down} =	0 lb	Utilization 45%	
Vertical uplift	20	V _{uplift} =	20 lb	Utilization 0%	

Determine pier embedment to resist lateral loads

Footing shape **Round**
Round diameter **36.0** inches

Diameter	b =	3.00 ft	[36.0"]	3'-0"	[1810.2.2]
Distance from ground to point of application "P"	h =	8.28 ft	(Moment / Lateral force)		

Ground surface condition **Nonconstrained**

Trial and Error - Depth of embedment	d _{trial} =	8.81 ft	
Δ = 0.00 ft			
Minimum embedment depth	d _{min} =	8.81 ft	[105.7"] 8'-9 5/8 "
Use depth of embedment	d =	8.83 ft	[106.0"] 8'-10"
Footing total height	L =	8.83 ft	[106.0"] 8'-10" [1810.2.2]

Minimum pier embedment depth calculations [1807.3.2]

Nonconstrained	d =	0.5A {1 + [1 + (4.36 h/A)] ^{1/2} }	where, A = 2.34 P / (S ₁ b)	[1807.3.2.1]
	S ₁ =	588 psf	lateral bearing pressure x embedment depth (max. 12ft) / 3	
	A =	4.35		
	d _{min} =	8.81 ft	[105.7"] 8'-9 5/8 "	
Constrained	d =	$\sqrt{4.25 P h / (S_3 b)}$		[1807.3.2.2]
	S ₃ =	1763 psf	lateral bearing pressure x embedment depth (max. 12ft)	
	d _{min} =	4.67 ft	[56.1"] 4'-8 0/1 "	

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Pier load capacity

Round	b =	3.00	ft	[36.0"]	3'-0"	
Pier perimeter		9.43	ft	[113.1"]		
Pier bearing area		7.07	ft ²	1018	in ²	
Effective skin area		83.29	ft ²	Embedment, excluding upper	0.0	feet
Pier volume		62.46	ft ³	2.313	cubic yard	
Gross pier weight		9370	lb	Unit weight of concrete	150	lb/ft ³
Net pier weight for compressive load		9370	lb	Conc. - Soil weight	0	150 lb/ft ³
Net pier weight for tensile load		5434	lb	Bouyant weight of concrete	87	lb/ft ³
Counterweight		11946	lb-ft	Pier weight x moment arm, which (0.85 x 1/2 pier diameter)		
Total bearing load		9370	lb	Vertical downward + pier weight		
Bearing resistance		10607	lb	Bearing area x bearing pressure capacity		
Skin friction resistance		20821	lb	AFR x effective skin area		
Allowable downward resistance		20821	lb	Maximum of bearing resistance or skin friction resistance		
Utilization		45%	ok!			
Allowable uplift resistance		26256	lb	(Pier weight + skin friction resistance)		
Utilization		0%	ok!			

Pier concrete reinforcements

Design Criteria

Soil Site Class	D	D	
Deep foundation type		Cast-in-place	[1810.3.9]
Seismic Design Category	2	B	B
Min. Concrete Strength	2500	f _c =	3000 psi [1808.8.1]
Reinforced Rebar Yield Strength		f _y =	60 ksi

Longitudinal bars

Bar size		#6 (3/4")	[1810.3.9.4.1]
Bar diameter	d _b =	0.750	in
Bar area	A _{s1} =	0.442	in ²
Number of bars	12	12	bar(s)
Total bar area	A _s =	5.30	in ² ok!
Min. reinforcement	0.0025	0.005	pier area
	A _{s min} =	5.09	in ²
Concrete cover		2.50	in [1808.8.2]
Rebar spacing	s =	8.12	in ok!
Min. spacing		1.50	in [ACI-318, 7.6.3.]

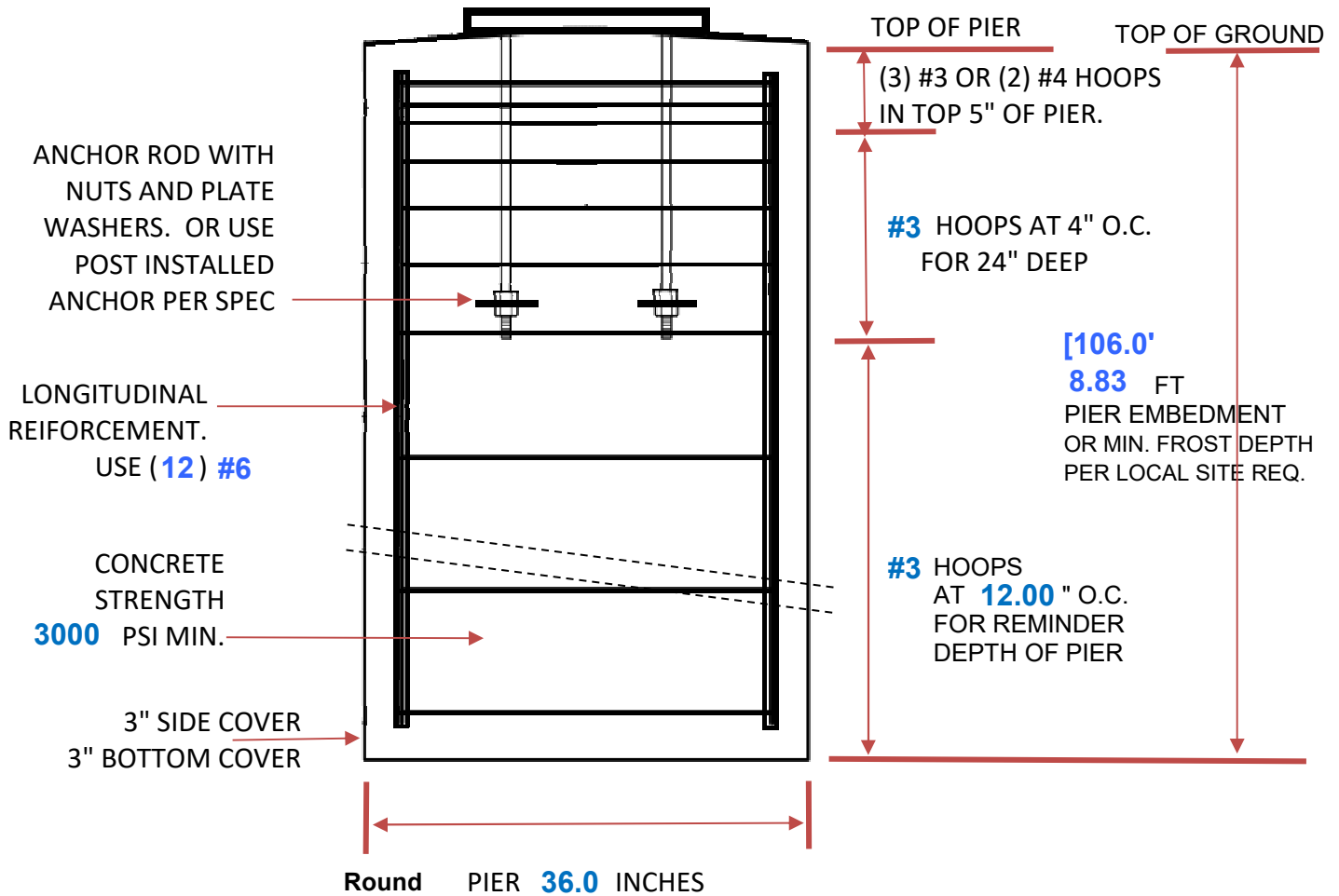
Transverse Ties

Bar size	#3 (3/8")	#3 (3/8")	#3 (3/8") #4 (1/2")
Bar diameter		d _b =	0.375 in
Top Ties	(3) #3 OR (2) #4	hoops in top 5" of Pier	
Tie spacing witin 24" deep from top ties		4.00	in o.c.
Reminding Ties Minimum Spacing		12.00	in

12.00

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Drilled Pier Sketch

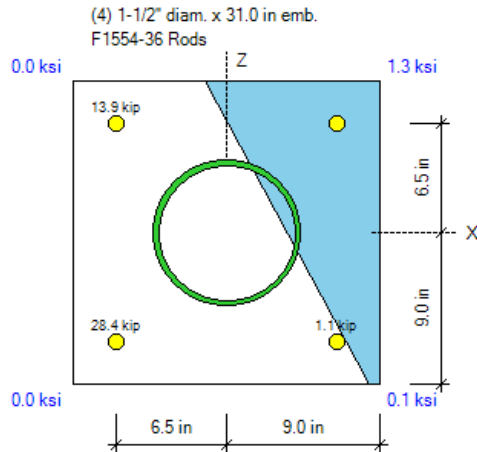


Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc. * S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project No :	10305-1	SS26005
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Project Name :	Ty Warner Park	
Phone:	(972) 896-5373	Location :	700 Blackhawk Drive Westmont IL 60559	
Email:	yurianto@modularconsultant.com	Subject :	Custom Sail SAIL	50'x20'x12'
		Date:	1/29/2026	Final

Base Plate & Anchor

ID : C3

Load Combination **IBC Section 1605.2 (LRFD)**



PLAN

Design Ratios	
Bearing	0.40 ✓
Anchorage	0.79 ✓

GEOMETRY

	Width	Length
Pipe8STD		
Steel Column	8.6	8.6 in
Steel Plate	18.0	18.0 in
Concrete Support	Wp1 18.0	Lp1 18.0 in
	Wp2 18.0	Lp2 18.0 in
Rod Edge	2.50	2.50 in
Thickness of Grout	1.0	in

DIMENSIONS ARE OK

FACTORED LOADS (LRFD)

Vertical Load P	-0.2	kip
Moment Mx	23.5	Mz ... 45.5 k-ft
Shear Vx	3.8	Vz ... -2.0 kip
Eccentr. ex	2730.0	ez ... 1410.0 in

BEARING STRESSES

Concrete Strength f'c	3.0	ksi
Minimum Stress	0.0	ksi
Maximum Stress fp	1.3	ksi
Bearing Strength ϕF_{pn}	3.3	ksi
fp / ϕF_{pn} Bearing Design Ratio	0.40	✓

BEARING STRESSES ARE OK

BASE PLATE DESIGN

Steel Strength Fy	36.0	ksi
<i>Plate Allowed to Work in the Plastic Range</i>		
Maximum Plate Moment	14.2	k-in/in
Thickness Required	1.32	in
Use Plate Thickness	1.50	in ✓

ANCHORAGE DESIGN

Anchor Rod Material	F1554-36
(4) Rods , fya = 36.0 ksi, futa = 58.0 ksi	
Rod Size	1-1/2" diam. x 31.0 in emb.
- Tension	
Total Tension Force N	43.3 kip
Max. Tension per Rod	28.4 kip
N / ϕN_n Tension Design Ratio	0.78 ✓
- Shear	
Shear Taken by	Anchor Rods only
Total Shear Force V	4.3 kip
Shear Force per Rod Vi	2.1 kip
<i>Only Front Rods Are Effective</i>	
<i>Anchor Reinforcement Provided</i>	
V / ϕV_n Shear Design Ratio	0.11 ✓
- Tension-Shear Interaction	
Combined Stress Ratio	0.79 ✓

ANCHORAGE DESIGN IS OK

Engineer :	Yurianto Yurianto, S.E.*, P.E., M.Sc.	Project No :	10305-1	SS26005
	* S.E. in State of IL, NV, HI, OK, AZ, MA, GA.	Project Name :	Ty Warner Park	
Address :	5760 Legacy Dr. B3-333. Plano, TX 75024	Location :	700 Blackhawk Drive Westmont IL 60559	
Phone:	(972) 896-5373	Subject :	Custom Sail SAIL	50'x20'x12'
Email:	yurianto@modularconsultant.com	Date:	1/29/2026	Final

Selected Support Reaction Governing Case R4 **Wind at 45 degrees** **RWIND**

Column Label **C4**

Node No. **4**

Reaction Line ID	Column Label	Node No.	Load Combination IBC Section 1605.3.1 (ASD)						Load Combination IBC Section 1605.2 (LRFD)					
			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)			Support Forces (lb) (+) Uplift			Support Moment (lb-ft)		
			P _X	P _Y	P _Z	M _X	M _Y	M _Z	P _X	P _Y	P _Z	M _X	M _Y	M _Z
R4-4	C4	4	-1809	-1517	-5	17928	-21967	0	-2426	-2062	62	24279	-29541	0

Drilled Piers ID : **C4**

Load combination **IBC Section 1605.3.1 (ASD)**

Trial and Error - Counterweight	10368		10367 lb-ft		[Calcs next page]
Applied Moment	17987	M =	17987 lb-ft	M = $\sqrt{M_x^2 + M_y^2}$ - Counterweight	
Applied Lateral force	2361	P =	2361 lb	P = $\sqrt{P_x^2 + P_y^2}$	
Vertical downward	5	V _{down} =	5 lb	Utilization 45%	
Vertical uplift	0	V _{uplift} =	0 lb	Utilization 0%	

Determine pier embedment to resist lateral loads

Footing shape **Round**
Round diameter **36.0** inches

Diameter	b =	3.00 ft	[36.0"]	3'-0"	[1810.2.2]
Distance from ground to point of application "P"	h =	7.62 ft	(Moment / Lateral force)		

Ground surface condition **Nonconstrained**

Trial and Error - Depth of embedment	d _{trial} =	7.60 ft			
Δ = 0.00 ft					
Minimum embedment depth	d _{min} =	7.60 ft	[91.2"]	7'-7 1/8 "	
Use depth of embedment	d =	7.67 ft	[92.0"]	7'-8"	
Footing total height	L =	7.67 ft	[92.0"]	7'-8"	[1810.2.2]

Minimum pier embedment depth calculations [1807.3.2]

Nonconstrained	d =	0.5A {1 + [1 + (4.36 h/A)] ^{1/2} }	where, A = 2.34 P / (S ₁ b)	[1807.3.2.1]
	S ₁ =	507 psf	lateral bearing pressure x embedment depth (max. 12ft) / 3	
	A =	3.63		
	d _{min} =	7.60 ft	[91.2"]	7'-7 1/8 "
Constrained	d =	$\sqrt{4.25 P h / (S_3 b)}$		[1807.3.2.2]
	S ₃ =	1520 psf	lateral bearing pressure x embedment depth (max. 12ft)	
	d _{min} =	4.09 ft	[49.1"]	4'-1 1/8 "

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Pier load capacity

Round	b =	3.00	ft	[36.0"]	3'-0"	
Pier perimeter		9.43	ft	[113.1"]		
Pier bearing area		7.07	ft ²	1018	in ²	
Effective skin area		72.29	ft ²	Embedment, excluding upper	0.0	feet
Pier volume		54.21	ft ³	2.008	cubic yard	
Gross pier weight		8132	lb	Unit weight of concrete	150	lb/ft ³
Net pier weight for compressive load		8132	lb	Conc. - Soil weight	0	150 lb/ft ³
Net pier weight for tensile load		4717	lb	Bouyant weight of concrete	87	lb/ft ³
Counterweight		10368	lb-ft	Pier weight x moment arm, which (0.85 x 1/2 pier diameter)		
Total bearing load		8137	lb	Vertical downward + pier weight		
Bearing resistance		10607	lb	Bearing area x bearing pressure capacity		
Skin friction resistance		18071	lb	AFR x effective skin area		
Allowable downward resistance		18071	lb	Maximum of bearing resistance or skin friction resistance		
Utilization		45%	ok!			
Allowable uplift resistance		22788	lb	(Pier weight + skin friction resistance)		
Utilization		0%	ok!	(No uplift)		

Pier concrete reinforcements

Design Criteria

Soil Site Class	D	D	
Deep foundation type		Cast-in-place	[1810.3.9]
Seismic Design Category	2	B	B
Min. Concrete Strength	2500	f _c =	3000 psi [1808.8.1]
Reinforced Rebar Yield Strength		f _y =	60 ksi

Longitudinal bars

Bar size		#6 (3/4")	[1810.3.9.4.1]
Bar diameter	d _b =	0.750	in
Bar area	A _{s1} =	0.442	in ²
Number of bars	12	12	bar(s)
Total bar area	A _s =	5.30	in ² ok!
Min. reinforcement	0.0025	0.005	pier area
		A _{s min} =	5.09 in ²
Concrete cover		2.50	in [1808.8.2]
Rebar spacing	s =	8.12	in ok!
Min. spacing		1.50	in [ACI-318, 7.6.3.]

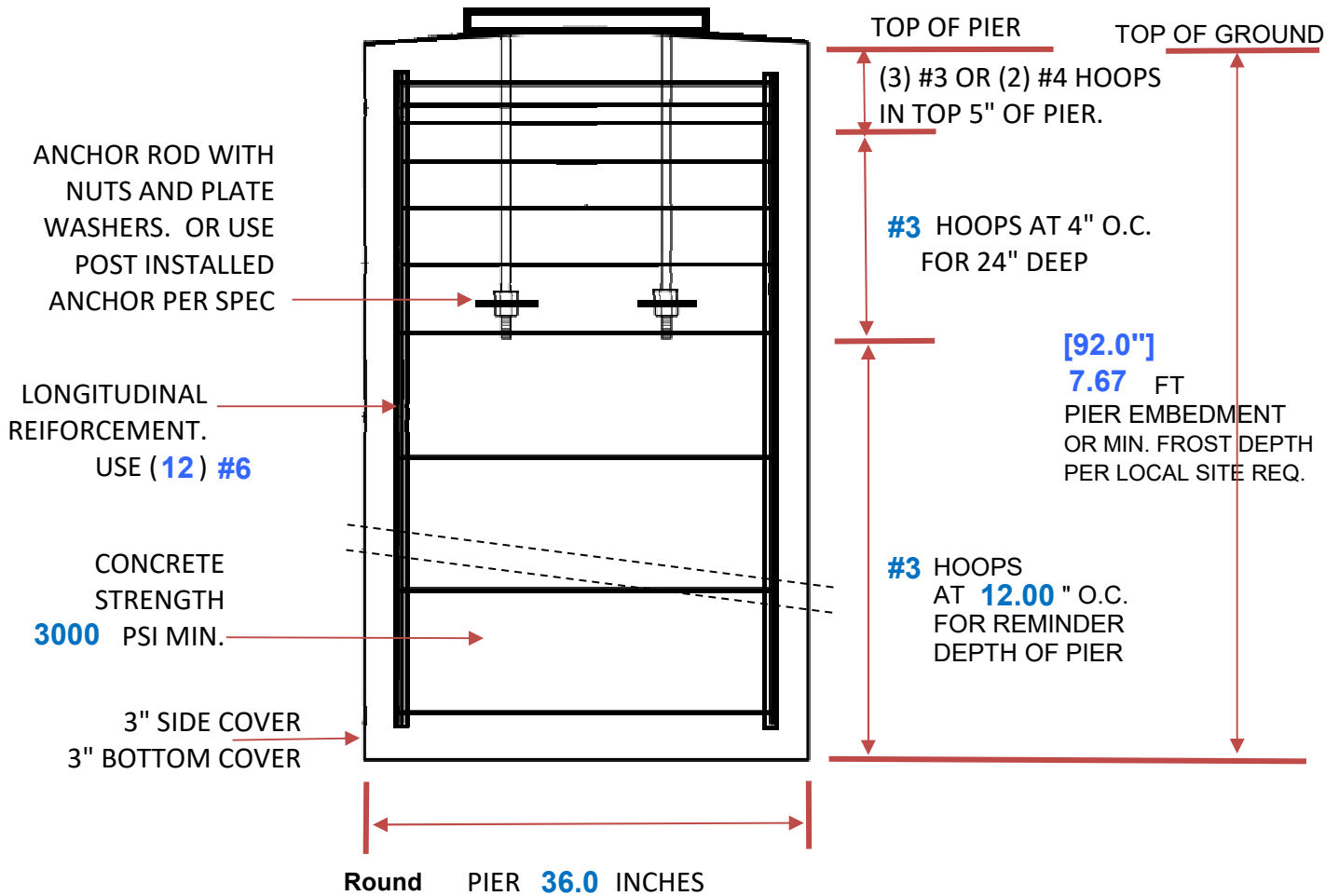
Transverse Ties

Bar size	#3 (3/8")	#3 (3/8")	#3 (3/8") #4 (1/2")	[1810.3.9.4.1]
Bar diameter		d _b =	0.375	in
Top Ties	(3) #3 OR (2) #4	hoops in top 5" of Pier		
Tie spacing witin 24" deep from top ties		4.00	in o.c.	
Reminding Ties Minimum Spacing		12.00	in	

12.00

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Drilled Pier Sketch

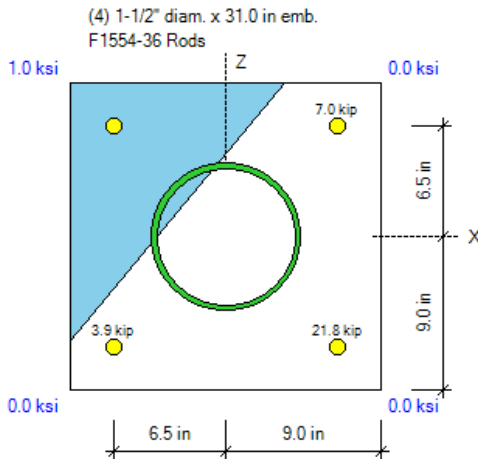


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Base Plate & Anchor

ID : C4

Load Combination **IBC Section 1605.2 (LRFD)**



PLAN

Design Ratios	
Bearing	0.31 ✓
Anchorage	0.60 ✓

GEOMETRY

	Width	Length
Pipe8STD		
Steel Column	8.6	8.6 in
Steel Plate	18.0	18.0 in
Concrete Support	Wp1 18.0 Wp2 18.0	Lp1 18.0 Lp2 18.0 in
Rod Edge	2.50	2.50 in
Thickness of Grout	1.0	in

DIMENSIONS ARE OK

FACTORED LOADS (LRFD)

Vertical Load P		-0.1	kip
Moment Mx	24.3	Mz	-29.5 k-ft
Shear Vx	-2.4	Vz	-2.1 kip
Eccentr. ex	3540.0	ez	2916.0 in

BEARING STRESSES

Concrete Strength f_c	3.0	ksi
Minimum Stress	0.0	ksi
Maximum Stress f_p	1.0	ksi
Bearing Strength ϕF_{pn}	3.3	ksi
$f_p / \phi F_{pn}$ Bearing Design Ratio	0.31	✓

BEARING STRESSES ARE OK

BASE PLATE DESIGN

Steel Strength F_y	36.0	ksi
<i>Plate Allowed to Work in the Plastic Range</i>		
Maximum Plate Moment	10.9	k-in/in
Thickness Required	1.16	in
Use Plate Thickness	1.50	in ✓

ANCHORAGE DESIGN

Anchor Rod Material	F1554-36
(4) Rods , $f_{ya} = 36.0$ ksi, $f_{uta} = 58.0$ ksi	
Rod Size	1-1/2" diam. x 31.0 in emb.
- Tension	
Total Tension Force N	32.7 kip
Max. Tension per Rod	21.8 kip
$N / \phi N_n$ Tension Design Ratio	0.59 ✓
- Shear	
Shear Taken by	Anchor Rods only
Total Shear Force V	3.2 kip
Shear Force per Rod V_i	1.6 kip
<i>Only Front Rods Are Effective</i>	
<i>Anchor Reinforcement Provided</i>	
$V / \phi V_n$ Shear Design Ratio	0.07 ✓
- Tension-Shear Interaction	
Combined Stress Ratio	0.60 ✓
<i>ANCHORAGE DESIGN IS OK</i>	