Your Project Calculations

Project Name: MichaelBugola|Option1|8panels-JB-RevD

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MichaelBugola|Option1|8panels-JB-RevD&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=ZCOOVEBIbIzZCFYMYLqeaivjwYkvS22hOanqM0p84lhD1rkyF2tft8Vg4IQm2jGH

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-45-L-4Hx2W-4FG0
Duty Classification:	SD
Module Width:	42.00 in
Module Length:	90.00in
Number of Rows:	4
Number of Columns:	2
Total Number of Modules:	8
Desired Tilt Angle:	30
Front Edge Clearance:	6
Total Array Height at Tilt:	13.04 ft
Total Frame Length:	15.00 ft
Frame Weight:	603 lbs
Array Dimensions N/S:	14.17 ft
Array Dimensions E/W:	15.17 ft
Rail Length:	170.00 in
Rail Spacing:	3.79 ft
Rail Check:	PASS (36% utilized)

Support Specifications

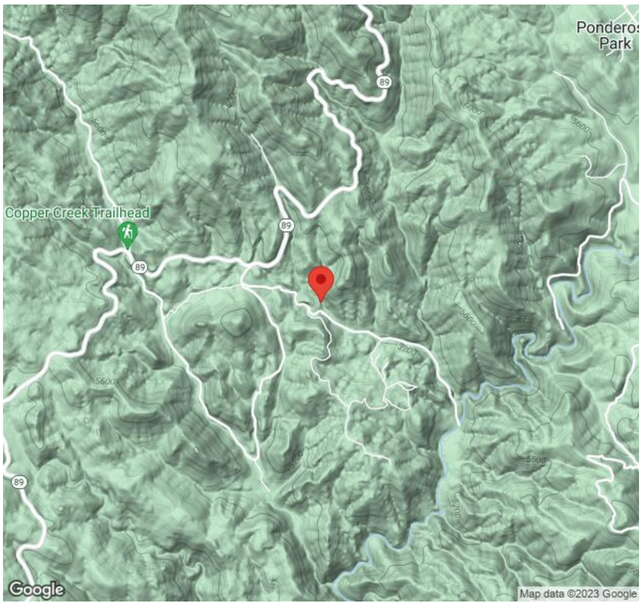
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.54 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø54 in
Foundation Depth (below grade):	Pile 1: 6.50 ft
Foundation Volume:	3.829 y^3
Foundation Result:	PASSED
Mount Twist:	0.000002 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	3030 FS Rd 73, Prescott, AZ 86303, USA
Wind Speed:	107 mph
Snow Load:	30 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.013196 ksf



Design Disclaimer

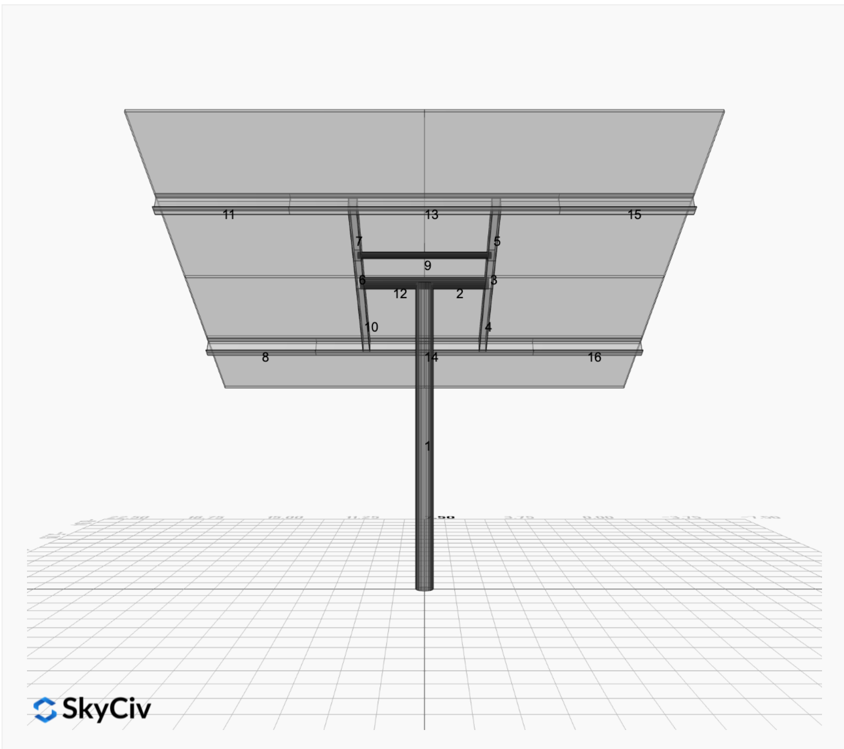
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

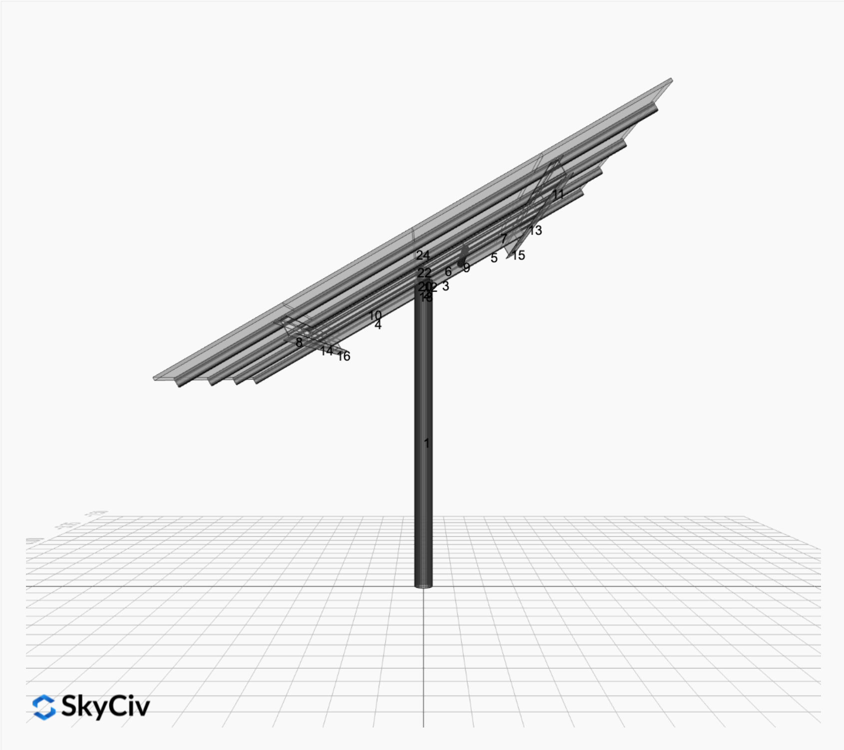
AutoDesigner Input

{"wind_speed_override":107,"snow_load_override":30,"direct_snow_load":false,"product_type":"Beam","project_id":"MichaelBugola|Option1|8panels-JB-RevD","site_address":"3030 FS Rd 73, Prescott, AZ 86303, USA","module_width":42,"module_length":90,"number_rows":"4","number_columns":2,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":30,"ground_clearance":6,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Round","foundation_size":54,"check_rails":true}

Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent

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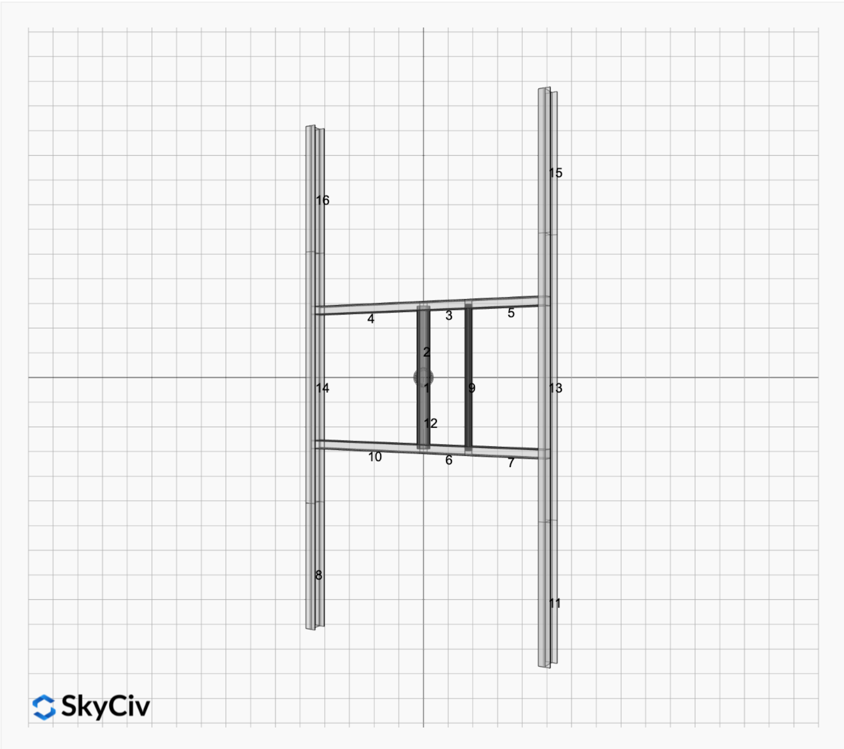
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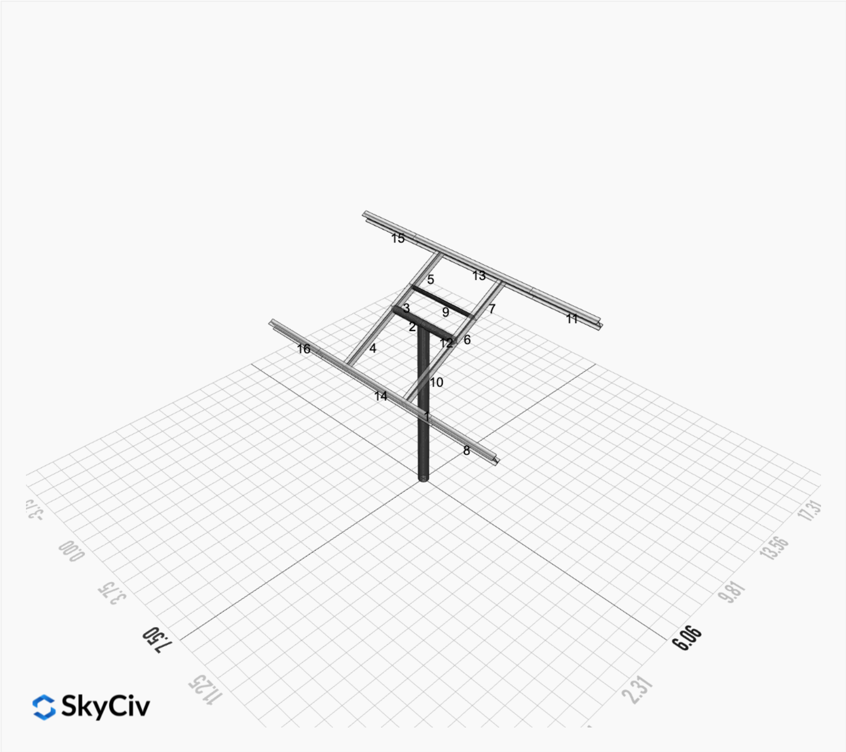
SkyCiv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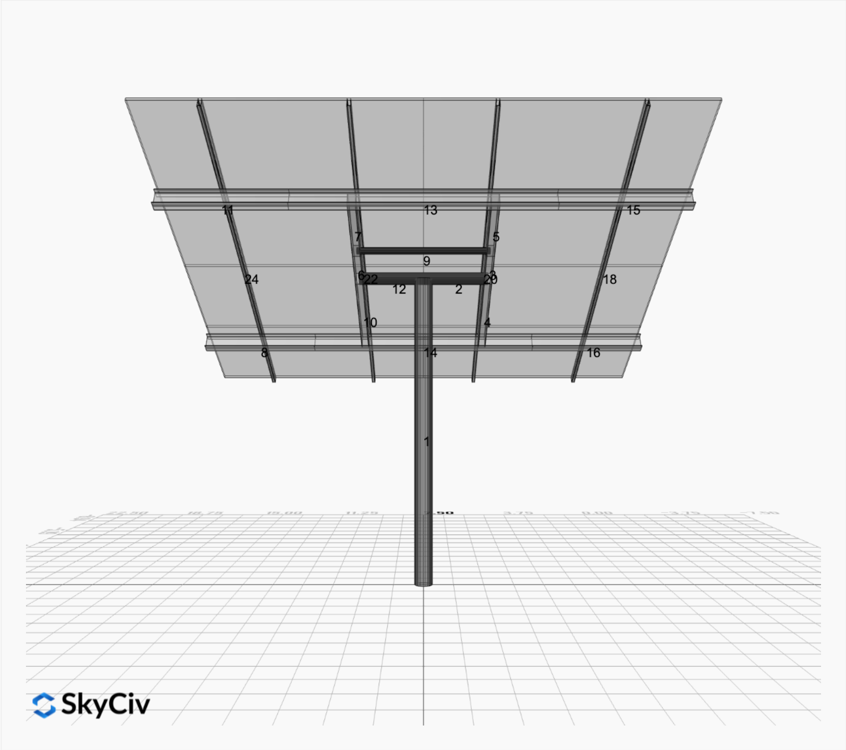
17.31
13.56
9.81
6.06
2.31
SkyCiv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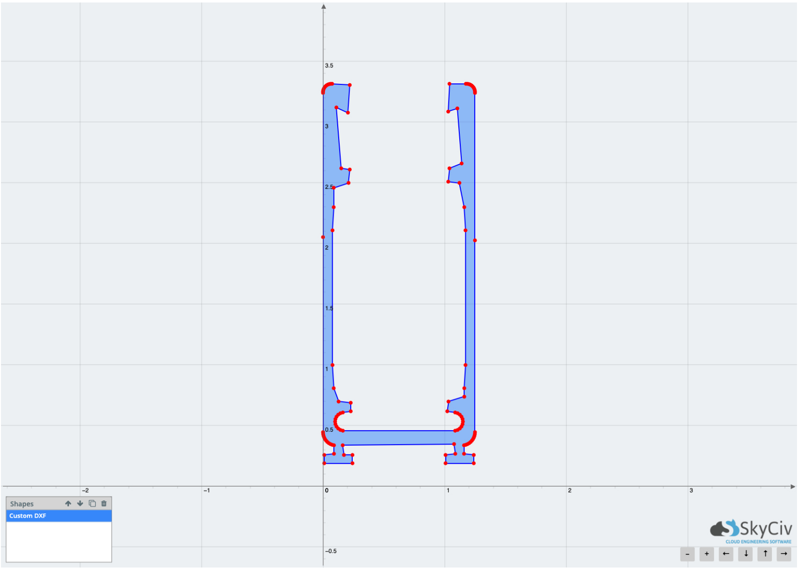
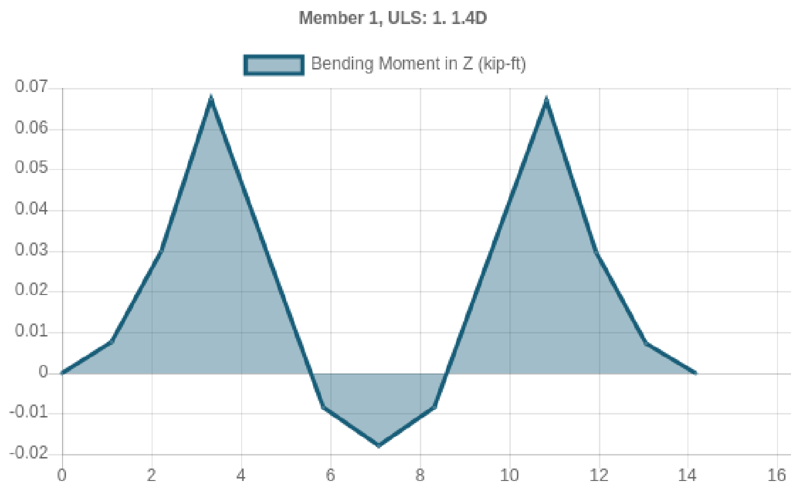
SkyCiv

Rail Design Check

Rail Length: 14.166666666666666 ft
Additional Restraints Required: None
Tributary Width: 3.7916666666666665 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0433 kip/ft
Snow (Y): -0.0250 kip/ft
Wind uplift Case A: 0.0685 kip/ft
Wind uplift Case A: 0.0685 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0951 kip/ft



Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	12.49438879	0.362	PASS
Material Yield	34.5	12.49438879	0.362	PASS
Material Strength	37	12.49438879	0.338	PASS



Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.5824	-0.0000	0.0000	-0.0000	0.0197
ULS: 2. D + L	0.0000	1.5824	-0.0000	0.0000	-0.0000	0.0197
ULS: 3. D + (S or Lr or R)	-0.0000	4.0108	-0.0000	0.0000	-0.0000	0.0291
ULS: 3. D + (S or Lr or R)	0.0000	1.5824	-0.0000	0.0000	-0.0000	0.0197
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.4037	-0.0000	0.0000	-0.0000	0.0268
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.5824	-0.0000	0.0000	-0.0000	0.0197
ULS: 5b. D + 0.7E	0.0000	1.5824	-0.0000	0.0000	-0.0000	0.0197
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.4037	-0.0000	0.0000	-0.0000	0.0268
ULS: 8. 0.6D + 0.7E	0.0000	0.9494	-0.0000	0.0000	-0.0000	0.0118
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3587	3.9358	-0.0000	0.0000	-0.0000	13.2633
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3587	3.9358	-0.0000	0.0000	-0.0000	13.2633
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1646	-0.4347	0.0000	0.0000	-0.0000	-10.8967
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9705	-0.0985	-0.0000	0.0000	-0.0000	-13.8494
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0190	5.1687	-0.0000	0.0000	-0.0000	9.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0190	5.1687	-0.0000	0.0000	-0.0000	9.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8735	1.8908	-0.0000	0.0000	-0.0000	-8.1606
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7279	2.1430	-0.0000	0.0000	-0.0000	-10.3751
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0190	3.3474	-0.0000	0.0000	-0.0000	9.9524
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0190	3.3474	-0.0000	0.0000	-0.0000	9.9524
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8735	0.0696	0.0000	0.0000	-0.0000	-8.1676
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7279	0.3217	-0.0000	0.0000	-0.0000	-10.3821
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3587	3.3028	-0.0000	0.0000	-0.0000	13.2555
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3587	3.3028	-0.0000	0.0000	-0.0000	13.2555
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1646	-1.0677	0.0000	0.0000	-0.0000	-10.9046
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9705	-0.7315	-0.0000	0.0000	-0.0000	-13.8573

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.7455
Shear X	-2.2645
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	23.5506

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	5.1687
Shear X	-1.3587
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	13.8573

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : MichaelBugola|Option1|8panels-JB-RevD
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **3030 FS Rd 73, Prescott, AZ 86303, USA** categorized as **Exposure B** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Morever, the structure is classified as **Risk Category I**. The location is elevated at **5322.32 ft** above mean sea level.

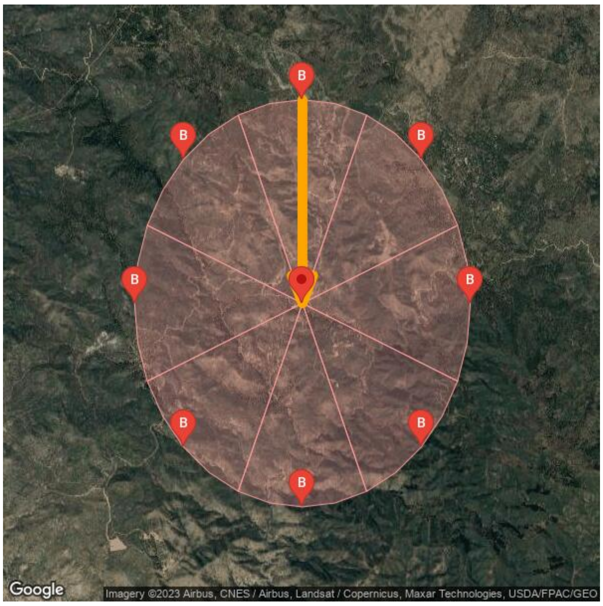


Figure 1. Site location.

Parameter	Value
Building Length, L	12.12 ft
Building Width, B	15.00 ft
Mean Roof Height, h	9.54 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.00°
Structure Type	Main Wind Force Resisting System (MWFRS)
Wind Blockage	Empty Under

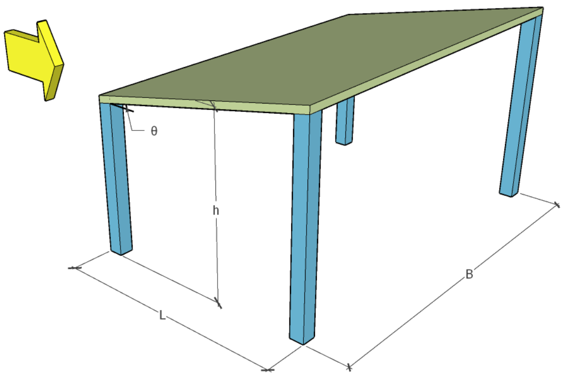


Figure 2. Building parameters.

REFERENCES	CALCULATIONS	RESULTS
Figure 26.5-1	**Basic Wind Speed, V** Wind speed for the address is **107 mph (defined by the user)** for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.	$V = 107$ mph **(defined by the user)**
Table 26.6-1	**Wind Directionality Factor, K_d** $K_d = 0.85$ - Wind Directionality Factor *For buildings*	$K_d = 0.85$
	Topographic Factor, K_{zt}	

Figure 26.8-1	$H/(L_h) = 0.25975$ *Slope of the hill, ridge, or escarpment*	$H/(L_h) = 0.25975$
Figure 26.8-1	$L_h = 964.47$ ft *Distance upwind of crest to where the difference in ground elevation is half the length of hill or escarpment.*	$L_h = 964.47$ ft
Figure 26.8-1	$H = 250.52$ ft *Height of the hill or escarpment relative to the upwind terrain*	$H = 250.52$ ft
Figure 26.8-1	$x = 8131.2$ ft *Distance (upwind or downwind) from the crest to the building site*	$x = 8131.2$ ft
Figure 26.8-1	$z = 0$ ft *Height above ground surface at building site.*	$z = 0$ ft
Figure 26.8-1	K_2 *Factor to account for reduction in speed-up with distance upwind or downwind of crest.* $K_2 = 1 - \frac{\lvert x \rvert}{\mu\, L_h}$ $K_2 = 0$ Where $x = 8131.2$ ft $\mu = \text{Horizontal attenuation factor} = 1.5$ $L_h = 964.47$ ft	
Figure 26.8-1	K_3 *Factor to account for reduction in speed-up with height above local terrain.* $K_3 = e^{-\gamma\, z/L_h}$ $K_3 = 1$ Where $z = 0$ ft $\gamma = \text{Height attenuation factor} = 4$ $L_h = 964.47$ ft	
Section 26.8	K_{zt} - Topographic Factor $K_{zt} = (1 + K_1\, K_2\, K_3)^2$ $K_{zt} = 1$ Where $K_1 = 0.24676$ $K_2 = 0$ $K_3 = 1$ **For calculating the topographic factor, the detected topography for the selected wind source direction is Hill.**	
	Ground Elevation Factor, K_e	
Section 26.9	K_e - Ground Elevation Factor $K_e = e^{-0.0000362\, E}$ $K_e = 0.82476$ Where $E = \text{Site Elevation} = 5322.3$ ft	
Section 26.9	$K_e = 0.82476$ - Ground Elevation Factor	$K_e = 0.82476$
	Velocity Pressure Exposure Coefficient, K_z	
Section 26.10	K_z - Velocity Pressure Exposure Coefficient *For $z < 15ft$* $K_z = 2.01 \times (15/z_g)^{2/\alpha}$	
Section 26.10	K_z - Velocity Pressure Exposure Coefficient *For $15ft \leq z \leq z_g$* $K_z = 2.01 \times (z/z_g)^{2/\alpha}$	
Section 26.10	K_z - Velocity Pressure Exposure Coefficient *For $z < 15ft$* $K_z = 2.01\, (15/z_g)^{2/\alpha}$ $K_z = 0.57472$ Where $z_g = 1200$ $\alpha = 7$	

Level	Elevation (ft)	K_z
h	9.542	0.575

	Velocity Pressure, q_h For the selected wind source direction.	
Section 26.10.2	q_h - Velocity Pressure at h $q_h = 0.00256\, K_z\, K_{zt}\, K_d\, K_e\, V^2$ $q_h = 11.809$ psf Where $K_z = \text{Velocity Pressure Exposure Coefficient} = 0.57472$ $K_{zt} = \text{Topographic Factor} = 1$ $K_d = \text{Wind Directionality Factor} = 0.85$ $V = \text{Basic Wind Speed} = 107$ mi/h $K_e = \text{Ground Elevation Factor} = 0.82476$	
	Velocity Pressure for All Directions	
Section 26.8	K_{zt} - Topographic Factor	

Section 26.8 K_{zt} - Topographic Factor

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

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $15ft \leq z \leq z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Direction	Exposure Category	K_z @ $h = 9.542ft$	K_{zt}	K_e	V (mph)	q_h (psf)
N	B	0.575	1.000	0.825	107.000	11.809
NE	B	0.575	1.000	0.825	107.000	11.809
E	B	0.575	1.000	0.825	107.000	11.809
SE	B	0.575	1.000	0.825	107.000	11.809
S	B	0.575	1.000	0.825	107.000	11.809
SW	B	0.575	1.000	0.825	107.000	11.809
W	B	0.575	1.000	0.825	107.000	11.809
NW	B	0.575	1.000	0.825	107.000	11.809

Net Pressure Coefficients, C_N

Figure 27.3-4 to 27.3-7 The net pressure coefficients, C_N, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.

Direction	Surface	C_N Case A	C_N Case B
0	Windward	-1.800	-2.500
	Leeward		-0.500
180	Windward	2.100	2.600
	Leeward		1.000
90	≤ h from windward edge	-0.800	0.800
	h to 2h from windward edge	-0.600	0.500
	> 2h from windward edge	-0.300	0.300

Gust Effect Factor, G

Section 26.11.1 $G = 0.85$ - Gust Effect Factor

The structure is assumed to be rigid.

$G = 0.85$

Design Wind Pressures (MWFRS)

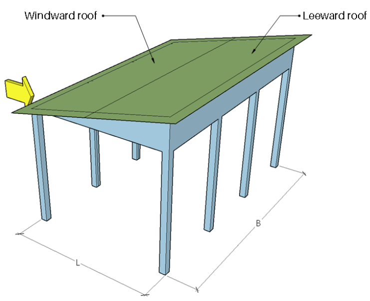
Section 27.3.2 p - Design Wind Pressure

For open buildings

$$p = q_h \times G \times C_N$$

For Wind Pressure - 0°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
0	Windward	11.809	0.850	-1.800	-2.500	-18.068	-25.094
	Leeward	11.809	0.850		-0.500	-18.068	-5.019
180	Windward	11.809	0.850	2.100	2.600	21.079	26.098
	Leeward	11.809	0.850		1.000	21.079	10.038



Wind along L - 0°

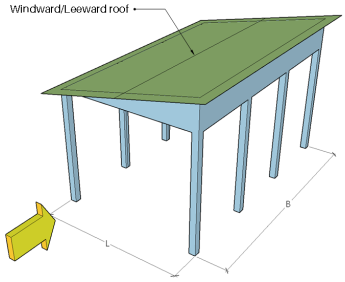
Service Wind Pressure - 0°/180°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	-10.841	-15.056
	Leeward	-10.841	-3.011
180	Windward	12.647	15.659
	Leeward	12.647	6.023

For Wind Pressure - 90°

Direction	Surface	q_h	G	C_N	C_N	p	p

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	11.809	0.850	-0.800	0.800	-8.030	8.030
	h to 2h from windward edge	11.809	0.850	-0.600	0.500	-6.023	5.019
	> 2h from windward edge	11.809	0.850	-0.300	0.300	-3.011	3.011



Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-4.818	4.818
	h to 2h from windward edge	-3.614	3.011
	> 2h from windward edge	-1.807	1.807

Section 27.3.2
Section 28.3.5

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted and be reduced based on the actual solidity ratio ϕ and number of frames n - See Figure 28.3-2.

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$

Section 27.3.2
Section 28.3.5

K_B - Frame Width Factor

For $L < 100ft$, $K_B = 1.8 - 0.01L$. Otherwise, $K_B = 0.8$.

$$K_B = 1.8 - 0.01 * L \leq 0.8$$

$$K_B = 1.6788$$

$$\text{Where } L = \text{Building Length} = 12.124 \text{ ft}$$

Section 28.3.5

K_S - Shielding Factor

$$K_S = 0.6 + 0.073 \times (n - 1) + \left(1.25 \times \phi^{1.8}\right)$$

Section 28.3.5

$K_S = 1$ - Shielding Factor — $K_S = 1$

Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n.

Figure 28.3-1
Section 28.3.5

$(GC_{pf})_{windward} = 0.4$ — $(GC_{pf})_{windward} = 0.4$

Using Zone 5 from Figure 28.3-1

Figure 28.3-1
Section 28.3.5

$(GC_{pf})_{leeward} = -0.29$ — $(GC_{pf})_{leeward} = -0.29$

Using Zone 6 from Figure 28.3-1

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \left[(GC_{pf})_{windward} - (GC_{pf})_{leeward}\right] K_B K_S$$

$$p = 13.679 \text{ psf}$$

REFERENCES	CALCULATIONS	RESULTS

Snow Load Detailed Calculations based on ASCE 7-16

Design Information :

Project Name : MichaelBugola|Option1|8panels-JB-RevD
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **3030 FS Rd 73, Prescott, AZ 86303, USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **5322 ft** above mean sea level.

Figure 1. Site location.

Additional details of the structure are shown in Table below and illustrated in Figure 2:

Parameter	Value
Building Length, L	12.124 ft
Building Width, B	15.000 ft
Mean Roof Height, h	9.542 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.000°

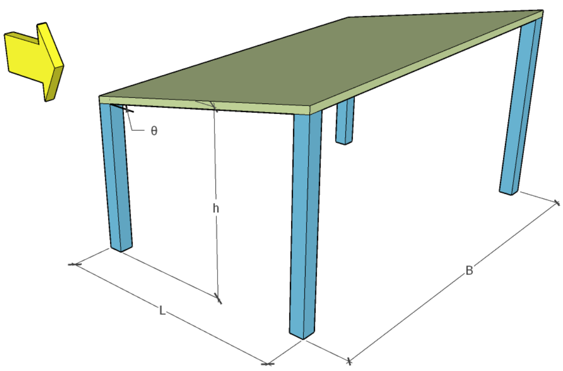


Figure 2. Building parameters.

Ground Snow Load, p_g

Section 7.2 of ASCE 7

The ground snow load, p_g, for the site location is **30 psf (defined by the user)** at elevation 5322.32 ft above mean sea leve

$p_g = 30$ psf **(defined b**

	The ground snow load, p_g, for the site location is **30 psf (defined by the user)** at elevation 5322.52 ft above mean sea level based on Section 7.2 of ASCE 7.	$p_g = 30$ psf **(defined by the user)**
Table 7-2 Section 7.3.1 of ASCE 7	**Exposure Factor, C_e** The exposure factor, C_e, for the structure is equal 0.90 as the terrain is categorized as **Exposure B** with exposure condition specified as **Fully Exposed** based on Table 7-2 Section 7.3.1 of ASCE 7.	$C_e = 0.90$
Table 7-3 Section 7.3.2 of ASCE 7	**Thermal Factor, C_e** Since the thermal condition of the structure is categorzied as **"Unheated and open air structures,"** the corresponding thermal factor, C_t, is equal 1.20 based on Table 7-3 Section 7.3.2 of ASCE 7.	$C_t = 1.20$
Table 1.5-2 of Chapter 1 ASCE 7	**Importance Factor, I_s** Since the structure is classified Risk Category I, the Importance Factor, I_s, is equal to **0.8**.	$I_s = 0.80$
Equation 7.3-1 of Section 7.3 ASCE 7	**Flat Roof Snow Load, p_f** The flat roof snow load, p_f, (psf) is calculated using the Equation 7.3-1: $p_f = 0.7C_eC_tI_sp_g$ $p_f = 0.7(0.90)(1.20)(0.80)(30.00) = 18.14psf$	$p_f = 18.14$ psf
Section 7.10 ASCE 7	**Rain-on-snow Surcharge Load, p_r** The rain-on-snow surcharge load, p_r, is equal to 0.00 psf since p_g > 20 psf.	$p_r = 0.00$ psf
Equation 7.7-1 of ASCE 7	**Snow Density, γ** The snow density, γ, is calculated using Equation 7.7-1 of ASCE 7 as: $\gamma = 0.13p_g + 14 \leq 30 = 0.13(30.00) + 14 \leq 30$ $\gamma = 17.90pcf$	$\gamma = 17.90pcf$
Section 7.4 ASCE 7	**Roof Slope Factor (Balanced), C_s** Since the roof is classified as cold roof ($C_t > 1.0$), the corresponding roof slope factor, C_s, is equal to 0.727 based on Figure 7.2c where $\theta = 30.00°$.	$C_s = 0.727$
Equation 7.4-1 of Section 7.4 ASCE 7	**Sloped Roof Snow Load (Balanced), p_s** The sloped roof snow load, p_s, (psf) is calculated using the Equation 7.4-1: $p_s = C_sp_f$ $p_s = (0.727)(18.14) = 13.20psf$	$p_s = 13.20$ psf

Project Details

Design Code: AISC 360-16 LRFD
Provision: LRFD
Country: United States

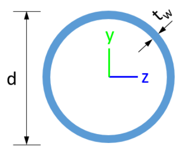
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

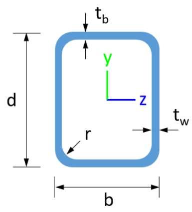
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

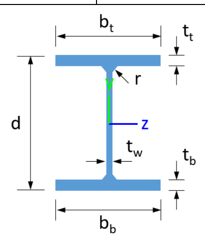
Section Dimensions



ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
7	6in Pipe Sch 40	6.63	0.28					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	7	20.04	20.04	9.54	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.76,1.67,1.67,1.68,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.49	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1
6	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1
8	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.76,1.67,1.67,1.68,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.49	300	200	1
11	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.08,1.04,1.04,1.04,1.06,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.09,1.04,1.04,1.04,1.05	300	200	1
14	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.19,1.04,1.04,1.04,2.80,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.04,1.04,1.04,1.01	300	200	1
15	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	108.60	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	54.44	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61

11	120.60	54.44	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	18.31	6.45	30.09	45.74
14	120.60	98.23	17.78	6.45	30.09	45.74
15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	23.36	6.45	30.09	45.74

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.071	0.557	0.000	0.030	0.000	0.565	#13	0.535	Not Required	Pass
2	0.002	0.398	0.151	0.088	0.026	0.521	#13	0.034	Not Required	Pass
3	0.009	0.600	0.092	0.061	0.010	0.697	#21	0.044	Not Required	Pass
4	0.009	0.596	0.203	0.060	0.025	0.742	#21	0.078	Not Required	Pass
5	0.009	0.372	0.210	0.060	0.030	0.423	#21	0.073	Not Required	Pass
6	0.009	0.600	0.092	0.061	0.010	0.697	#21	0.044	Not Required	Pass
7	0.009	0.372	0.210	0.060	0.030	0.423	#21	0.073	Not Required	Pass
8	0.000	0.070	0.098	0.029	0.007	0.168	#21	Not Required	Not Required	Pass
9	0.008	0.058	0.041	0.001	0.000	0.102	#21	0.198	Not Required	Pass
10	0.009	0.596	0.203	0.060	0.025	0.742	#21	0.078	Not Required	Pass
11	0.000	0.070	0.098	0.029	0.007	0.168	#21	Not Required	Not Required	Pass
12	0.002	0.398	0.151	0.088	0.026	0.521	#13	0.034	Not Required	Pass
13	0.004	0.200	0.211	0.042	0.011	0.402	#21	0.177	Not Required	Pass
14	0.004	0.204	0.211	0.042	0.011	0.402	#21	0.177	Not Required	Pass
15	0.000	0.070	0.098	0.029	0.007	0.168	#21	Not Required	Not Required	Pass
16	0.000	0.070	0.098	0.029	0.007	0.168	#21	Not Required	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

[illegible]	capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

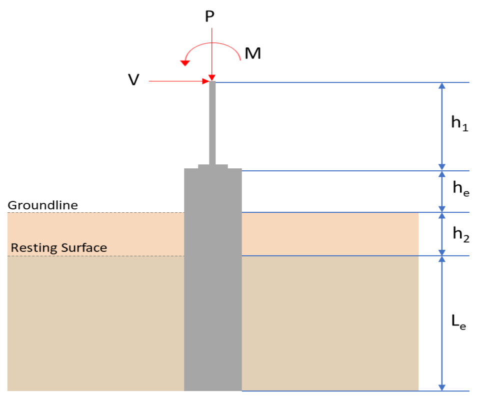
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: round
$D = 54\ \mathrm{in}$ - Pile diameter
$L = 6.5\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) $(\mathrm{psf/ft})$
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.169	7.745
V_x (kip)	-1.359	-2.265
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	13.857	23.551

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{D}$$

$$H_o = \frac{(-1.359\ \mathrm{kip})}{(54\ \mathrm{in})}$$

$$H_o = -0.302\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\ H)}{D}$$

$$M_o = \frac{(13.857 \text{ kipft}) + ((-1.359 \text{ kip}) \times (0 \text{ ft}))}{(54 \text{ in})}$$

$$M_o = 3.0793 \text{ kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,x} = 6.0012 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = MAX[(6.0012 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.001 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

$Ratio$ - Embedded depth

$$Ratio = \frac{L_{e,req}}{L_e}$$

$$Ratio = \frac{(6.001 \text{ ft})}{(6.5 \text{ ft})}$$

$$Ratio = 0.92323$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cros-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(54 \text{ in})}{2}\right)^2$$

$$A = 15.904 \text{ ft}^2$$

q - End-bearing pressure

$$q = \frac{P_v}{A}$$

$$q = \frac{(5.169 \text{ kip})}{(15.904 \text{ ft}^2)}$$

$$q = 0.32501 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

$$Ratio = \frac{q}{q_a}$$

$$Ratio = \frac{(0.32501 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.1625$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

$$L/D = \frac{L}{D}$$

$$L/D = \frac{(6.5 \text{ ft})}{(54 \text{ in})}$$

$$L/D = 1.4444$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.302 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 3.0793 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

$$a = \frac{(4\,M_o\,L_e) + \left(3\,H_o\,L_e^2\right)}{(6\,M_o) + (4\,H_o\,L_e)}$$

$$a = \frac{(4 \times (3.0793 \text{ kipft/ft}) \times (6.5 \text{ ft})) + \left(3 \times (-0.302 \text{ kip/ft}) \times (6.5 \text{ ft})^2\right)}{(6 \times (3.0793 \text{ kipft/ft})) + (4 \times (-0.302 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4949 \text{ ft}$$

p - Earth pressure against the pile at distance a/2 from resting surface,

$$p = \frac{1.178\,[(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\,[(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (3.0793 \text{ kipft/ft})) + (3 \times (-0.302 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.0793 \text{ kipft/ft})) + (2 \times (-0.302 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.2169 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\,[(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (3.0793 \text{ kipft/ft})) + ((-0.302 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.93596 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

p_a - Allowable lateral soil pressure at depth a/2,

$$p_a = R\,\frac{a}{2}$$

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4949 \text{ ft})}{2}$$

$$p_a = 0.33712 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

$$Ratio = \frac{p}{p_a}$$

$$Ratio = \frac{\left(0.2169 \text{ kip/ft}^2\right)}{\left(0.33712 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.6434$$

Status: **PASS**
Ratio: **0.640**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

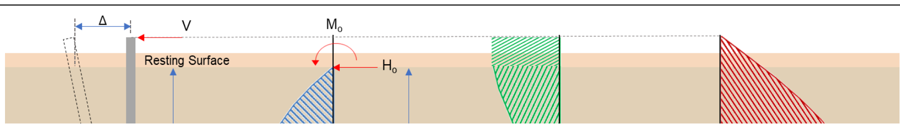
$Ratio$ - Lateral soil capacity

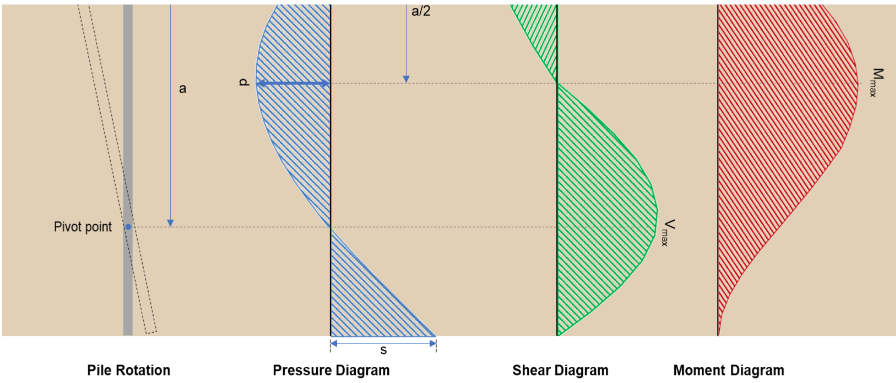
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.93596 \text{ kip/ft}^2\right)}{\left(0.975 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.95996$$

Status: **PASS**
Ratio: **0.960**





Shear force and Bending moment (x-direction, LRFD)

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{D}$$

$$H_o = \frac{(-2.265 \text{ kip})}{(54 \text{ in})}$$

$$H_o = -0.50333 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\, H)}{D}$$

$$M_o = \frac{(23.551 \text{ kipft}) + ((-2.265 \text{ kip}) \times (0 \text{ ft}))}{(54 \text{ in})}$$

$$M_o = 5.2336 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

$$E = \frac{M_o}{H_o}$$

$$E = \frac{(5.2336 \text{ kipft/ft})}{(-0.50333 \text{ kip/ft})}$$

$$E = 10.398 \text{ ft}$$

a - Distance from resting surface to pivot point,

$$a = \frac{(4\, M_o\, L_e) + \left(3\, H_o\, L_e^2\right)}{(6\, M_o) + (4\, H_o\, L_e)}$$

$$a = \frac{(4 \times (5.2336 \text{ kipft/ft}) \times (6.5 \text{ ft})) + \left(3 \times (-0.50333 \text{ kip/ft}) \times (6.5 \text{ ft})^2\right)}{(6 \times (5.2336 \text{ kipft/ft})) + (4 \times (-0.50333 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4927 \text{ ft}$$

V_{max} - Max shear force located at depth a,

$$V_{max} = (H_o\, D) \left[1 - \left[3\left(\frac{4\,E}{L_e} + 3\right)\left(\frac{a}{L_e}\right)^2\right] + \left[4\left(\frac{3\,E}{L_e} + 2\right)\left(\frac{a}{L_e}\right)^3\right]\right]$$

$$V_{max} = ((-0.50333 \text{ kip/ft}) \times (54 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.398 \text{ ft})}{(6.5 \text{ ft})} + 3\right) \times \left(\frac{(4.4927 \text{ ft})}{(6.5 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.398 \text{ ft})}{(6.5 \text{ ft})} + 2\right) \times \left(\frac{(4.4927 \text{ ft})}{(6.5 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 7.9049 \text{ kip}$$

M_{max} - Max bending moment located at depth a/2,

$$M_{max} = (H_o\, D\, L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2\,L_e}\right) - \left[\left(\frac{4\,E}{L_e} + 3\right)\left(\frac{a}{2\,L_e}\right)^3\right] + \left[\left(\frac{3\,E}{L_e} + 2\right)\left(\frac{a}{2\,L_e}\right)^4\right]\right]$$

$$M_{max} = ((-0.50333 \text{ kip/ft}) \times (54 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(10.398 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4927 \text{ ft})}{2 \times (6.5 \text{ ft})}\right) - \left[\left(\frac{4 \times (10.398 \text{ ft})}{(6.5 \text{ ft})} + 3\right) \times \left(\frac{(4.4927 \text{ ft})}{2 \times (6.5 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.398 \text{ ft})}{(6.5 \text{ ft})} + 2\right) \times \left(\frac{(4.4927 \text{ ft})}{2 \times (6.5 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 24.356 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5$ ksi - Concrete strength,
$f_{yk} = 60$ ksi - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 — $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 2290.2 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 — $A_{st,required}$

$$A_{st,required} = Min\left[\frac{\frac{P}{\phi\,\alpha} - \left(0.85\,f'_{ck}\,A_g\right)}{f_{yk} - \left(0.85\,f'_{ck}\right)}, (0.08\,A_g)\right]$$

$$A_{st,required} = Min\left[\frac{\frac{(7.745 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2290.2 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2290.2 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -83.848 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

$$A_{min} = Max\left[A_{st,required}, (0.0018\,A_g)\right]$$

$$A_{min} = Max\left[\left(-83.848 \text{ in}^2\right), \left(0.0018 \times \left(2290.2 \text{ in}^2\right)\right)\right]$$

$$A_{min} = 4.1224 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1224 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\,\frac{\pi\,d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

$Ratio$ - Capacity

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{\left(4.1224 \text{ in}^2\right)}{\left(4.2951 \text{ in}^2\right)}$$

$$Ratio = 0.95978$$

Status: **PASS**
Ratio: **0.960**

25.2.3 — s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = Max\left[1.5, (1.5\,d_{bar})\right]$$

$$s_{rebar} = Max\left[1.5, (1.5 \times (0.625 \text{ in}))\right]$$

$$s_{rebar} = 1.5 \text{ in}$$

Ties:

25.7.2.2 — Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 — s_{ties} - Maximum center-to-center spacing of ties,

$$s_{ties} = Min\left[(16\,d_{bar}), (48\,d_{ties}), D\right]$$

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (54 \text{ in})\right]$$

$$s_{ties} = 10 \text{ in}$$

Summary:

	Main reinforcement: **14 - #5 (0.625 in)** Ties: **#3(0.375 in) - 10 in**		
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.85\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2290.2\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2826.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(7.745\text{ kip})}{(2826.2\text{ kip})}$$ $$Ratio = 0.0027404$$	Status: **PASS** Ratio: **0.000**	
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 54\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (54\text{ in})$$ $$d = 43.2\text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(43.2\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.61314$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.61314) \times \sqrt{(2500\text{ psi})} \times (54\text{ in}) \times (43.2\text{ in})$$ $$V_{c,max} = 357.58\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 7.745\text{ kip} \rightarrow 7745\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.61314) \times \sqrt{(2500\text{ psi})} + \frac{(7745\text{ lbf})}{6 \times (2290.2\text{ in}^2)}\right] \times (54\text{ in}) \times (43.2\text{ in})$$ $$V_{c,a} = 144.35\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.61314) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (54\text{ in}) \times (43.2\text{ in})$$ $$V_{c,b} = 434.63\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(357.58\text{ kip}), (144.35\text{ kip}), (434.63\text{ kip})\right]$$		

$$V_c = 144.35\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$

$$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (54\ \text{in}) \times (43.2\ \text{in})$$

$$V_{s,a} = 933.12\ \text{kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$

$$A_v = 0.11045\ \text{in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (43.2\ \text{in})}{(10\ \text{in})}$$

$$V_{s,b} = 57.256\ \text{kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\,[V_{s,a}, V_{s,b}]$$

$$V_s = MIN\,[(933.12\ \text{kip}), (57.256\ \text{kip})]$$

$$V_s = 57.256\ \text{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\ (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((144.35\ \text{kip}) + (57.256\ \text{kip}))$$

$$\phi V_n = 131.04\ \text{kip}$$

Considering x-direction:

$V_{max} = 7.9049\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(7.9049\ \text{kip})}{(131.04\ \text{kip})}$$

$$Ratio = 0.060323$$

Status: **PASS**
Ratio: **0.060**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi\, D^3}{32}$$

$$S_m = \frac{\pi \times (54\ \text{in})^3}{32}$$

$$S_m = 15459\ \text{in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 15458.992in^3$$

$$\phi M_{n,1} = 209.341kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(15459\ \mathrm{in}^3\right)$$

$$\phi M_{n,2} = 1779.4\ \mathrm{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(209.34\ \mathrm{kipft}), (1779.4\ \mathrm{kipft})]$$

$$\phi M_n = 209.34\ \mathrm{kipft}$$

Considering x-direction:

$M_{max} = 24.356\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$Ratio = \frac{(24.356\ \mathrm{kipft})}{(209.34\ \mathrm{kipft})}$$

$$Ratio = 0.11634$$

Status: **PASS**
Ratio: **0.120**