Your Project Calculations

Project Name: MTSOLAR_77LC0HCD9B1J

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MTSOLAR_77LC0HCD9B1J&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=QQK8s1OlQTMsWj3fV0h0pV464YYjPYD0PcDaSf0AQ6v54rOt5YWTZS7VEfgsvpoB

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-12-L-3Hx2W-4A7G
Duty Classification:	SD
Module Width:	48.00 in
Module Length:	60.00in
Number of Rows:	3
Number of Columns:	2
Total Number of Modules:	6
Desired Tilt Angle:	65
Front Edge Clearance:	5
Total Array Height at Tilt:	15.93 ft
Total Frame Length:	9.50 ft
Frame Weight:	521 lbs
Array Dimensions N/S:	12.13 ft
Array Dimensions E/W:	10.17 ft
Rail Length:	145.50 in
Rail Spacing:	2.54 ft
Rail Check:	PASS (20% utilized)

Support Specifications

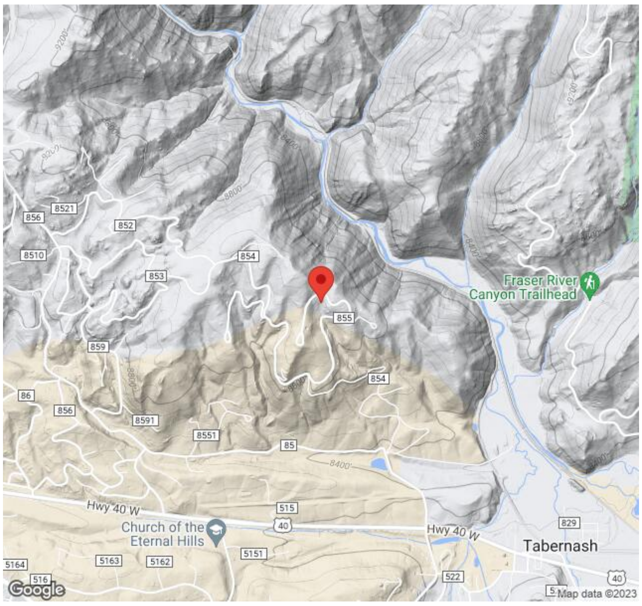
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.49 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.25 ft
Foundation Volume:	2.160 y^3
Foundation Result:	PASSED
Mount Twist:	0.000002 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	637 Rainbow, Tabernash, CO 80478, USA
Wind Speed:	115 mph
Snow Load:	90 psf
Design Uplift Pressure:	0.028318 ksf
Design Downforce Pressure:	-0.028318 ksf
Design Snow Pressure:	0.004948 ksf



Design Disclaimer

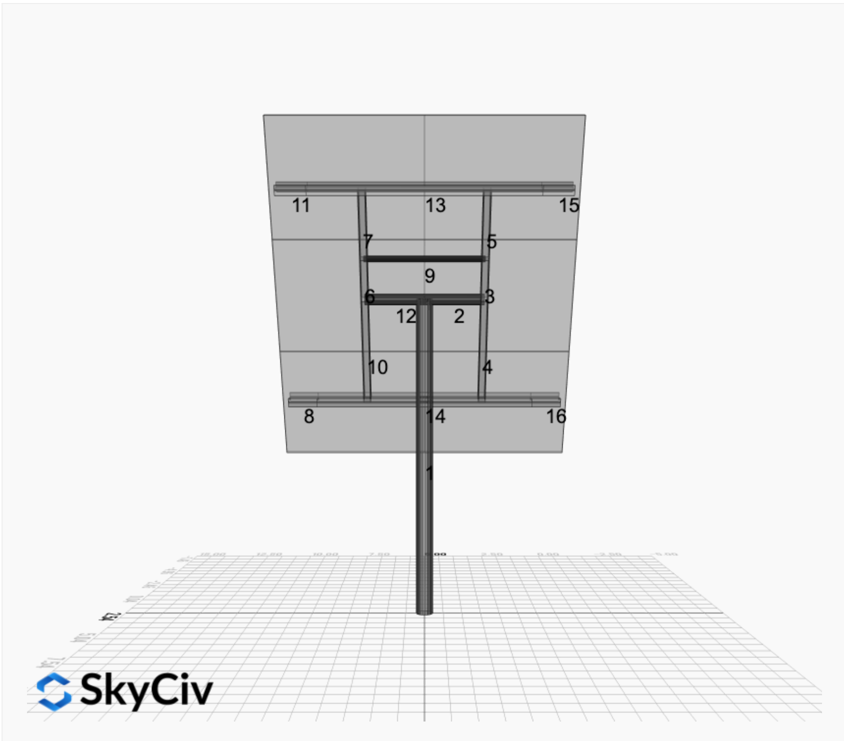
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

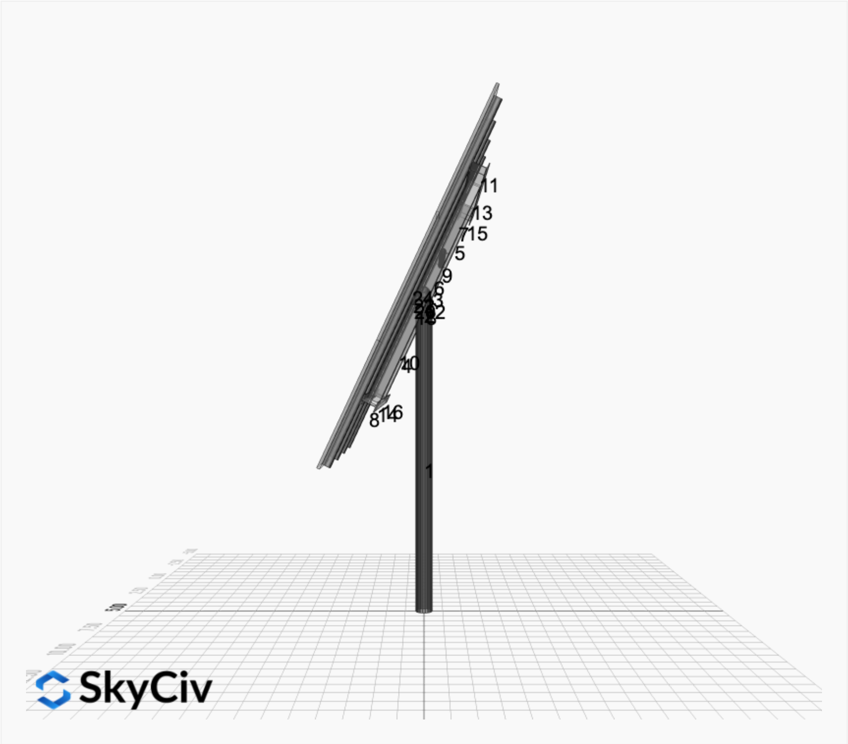
AutoDesigner Input


```
{"product_type":"Beam","project_id":"MTSOLAR_77LC0HCD9B1J","site_address":"637 Rainbow, Tabernash, CO 80478, USA","module_width":48,"module_length":60,"number_rows":"3","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":65,"ground_clearance":5,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","foundation_type":"Round","foundation_size":36,"check_rails":true,"wind_speed_override":115,"snow_load_override":90,"direct_snow_load":false}
```

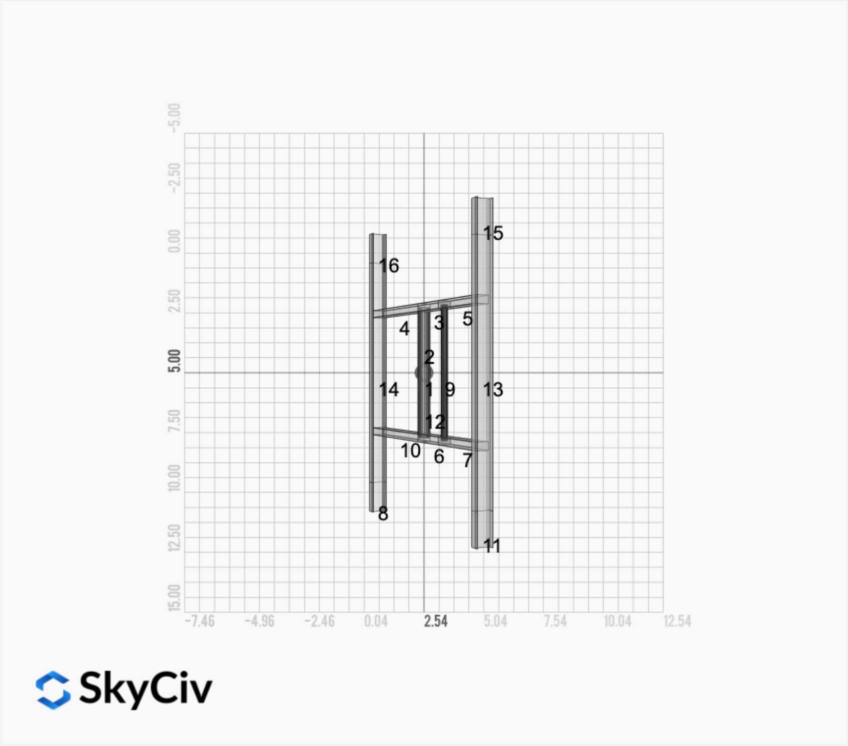

Design Notes:

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

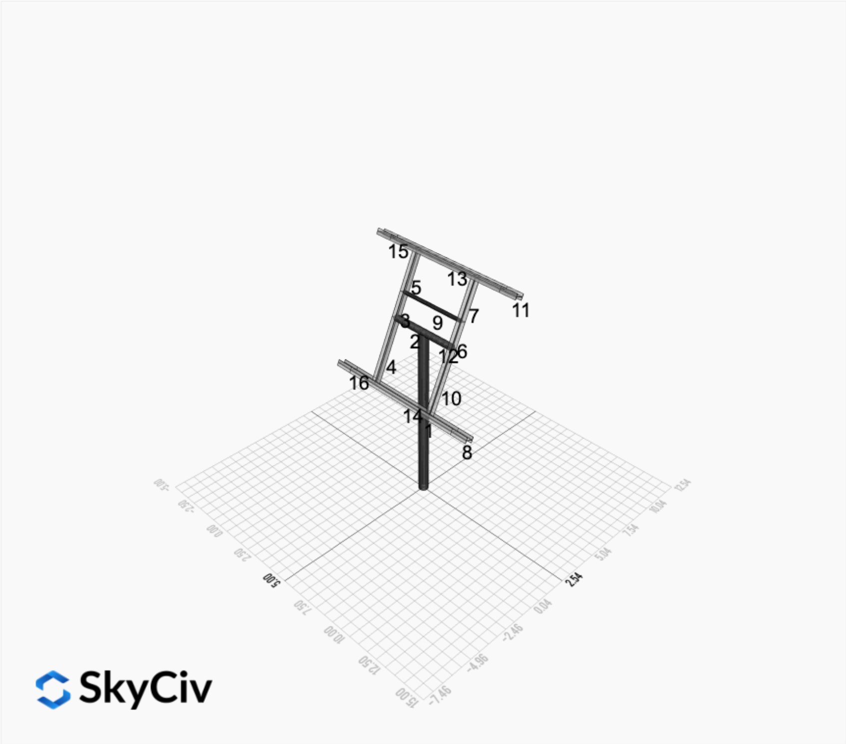


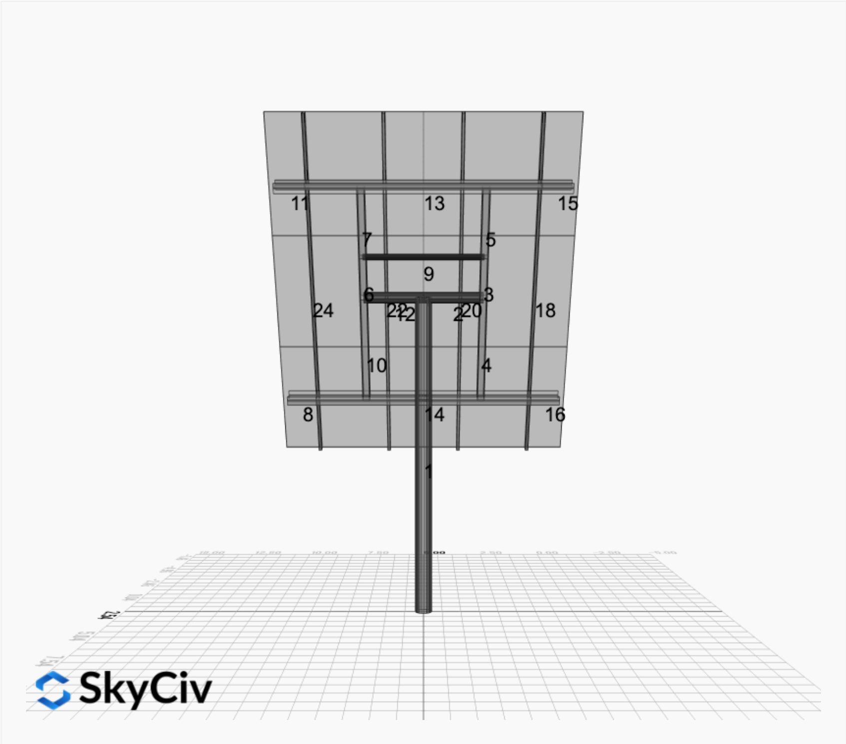


SkyCiv



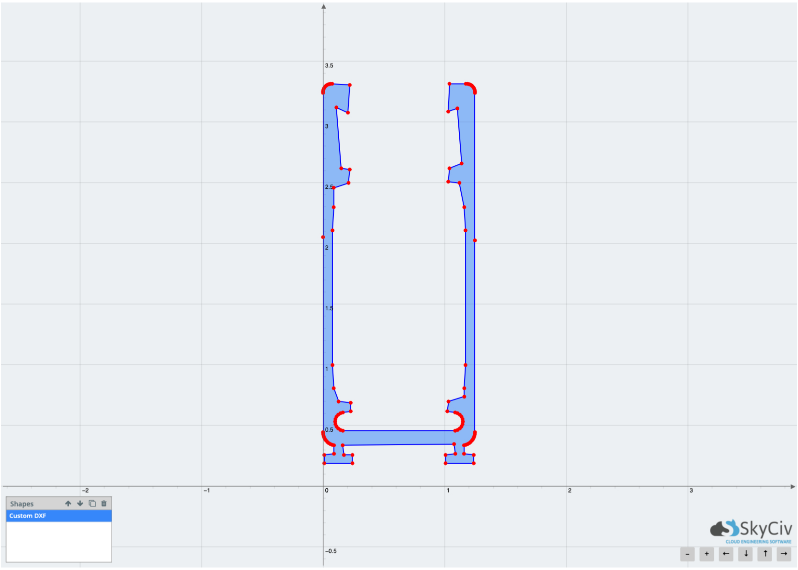
16
15
4
3
5
2
14
1
9
13
12
10
6
7
8
11
-5.00
-2.50
0.00
2.50
5.00
7.50
10.00
12.50
15.00
-7.46
-4.96
-2.46
0.04
2.54
5.04
7.54
10.04
12.54
SkyCiv


15
13
5
11
7
9
3
2
12
6
4
16
10
14
1
8
SkyCiv

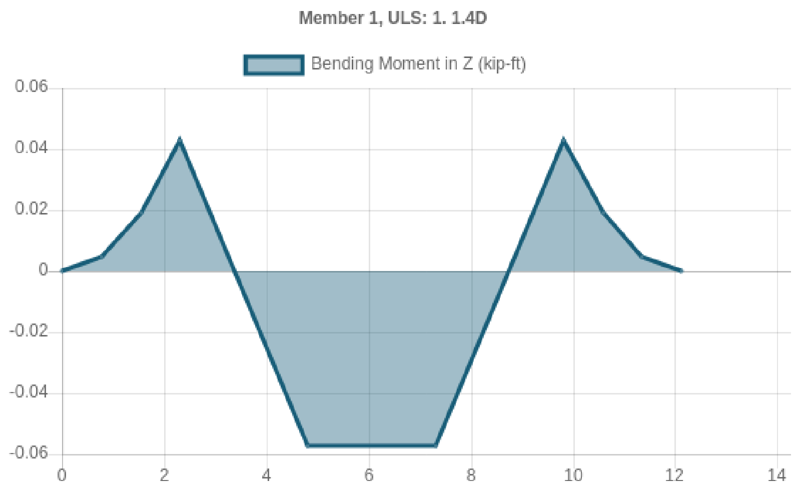

11
13
15
7
5
9
6
3
24
22
12
2
20
18
10
4
8
14
16
1
SkyCiv

Rail Design Check

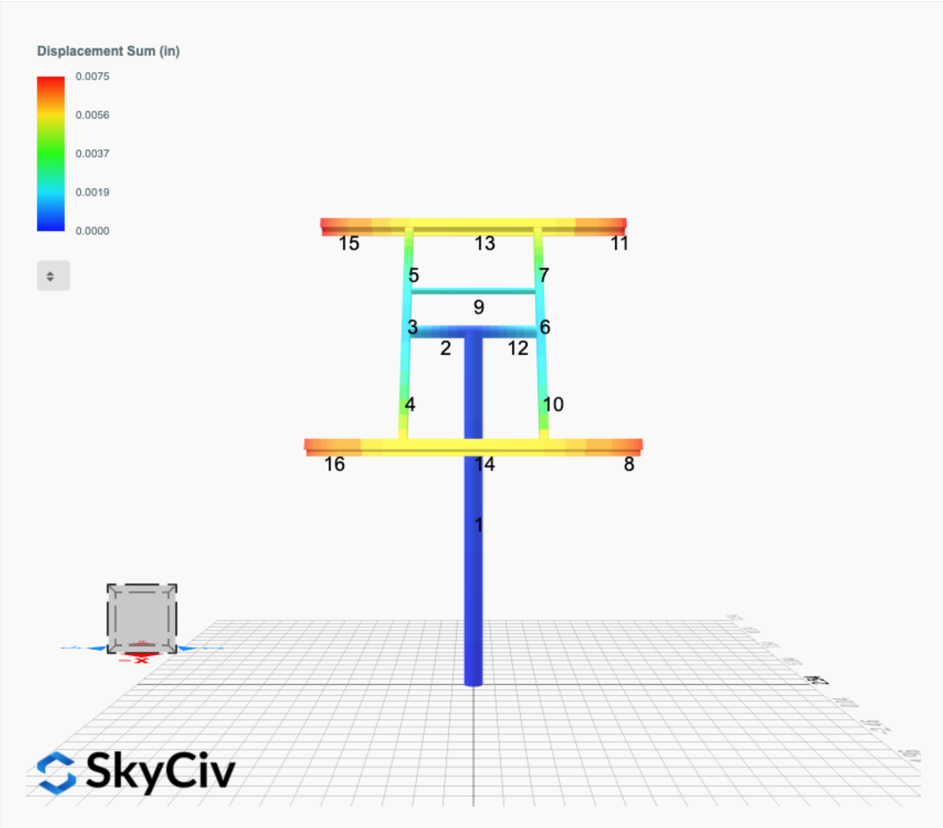
Rail Length: 12.125 ft
Additional Restraints Required: None
Tributary Width: 2.5416666666666665 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0053 kip/ft
Snow (Y): -0.0114 kip/ft
Wind uplift Case A: 0.0720 kip/ft
Wind downforce Case A: 0.0720 kip/ft
Dead (Panel load) (X): 0.0053 kip/ft
Dead (Panel load) (Y): -0.0115 kip/ft

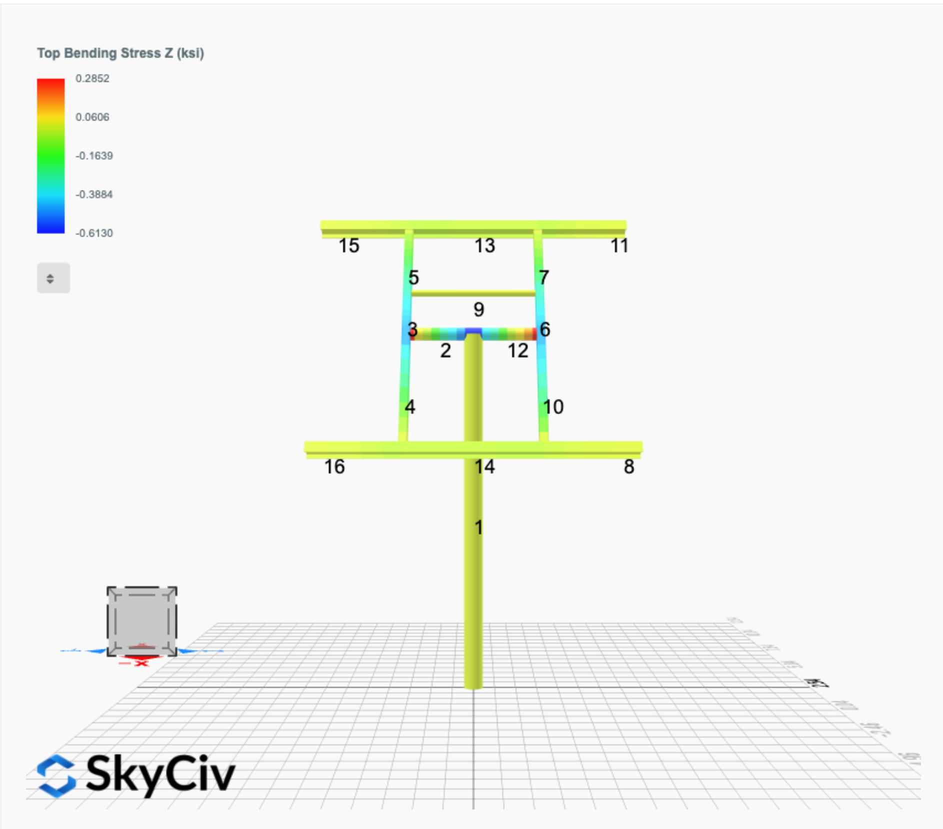


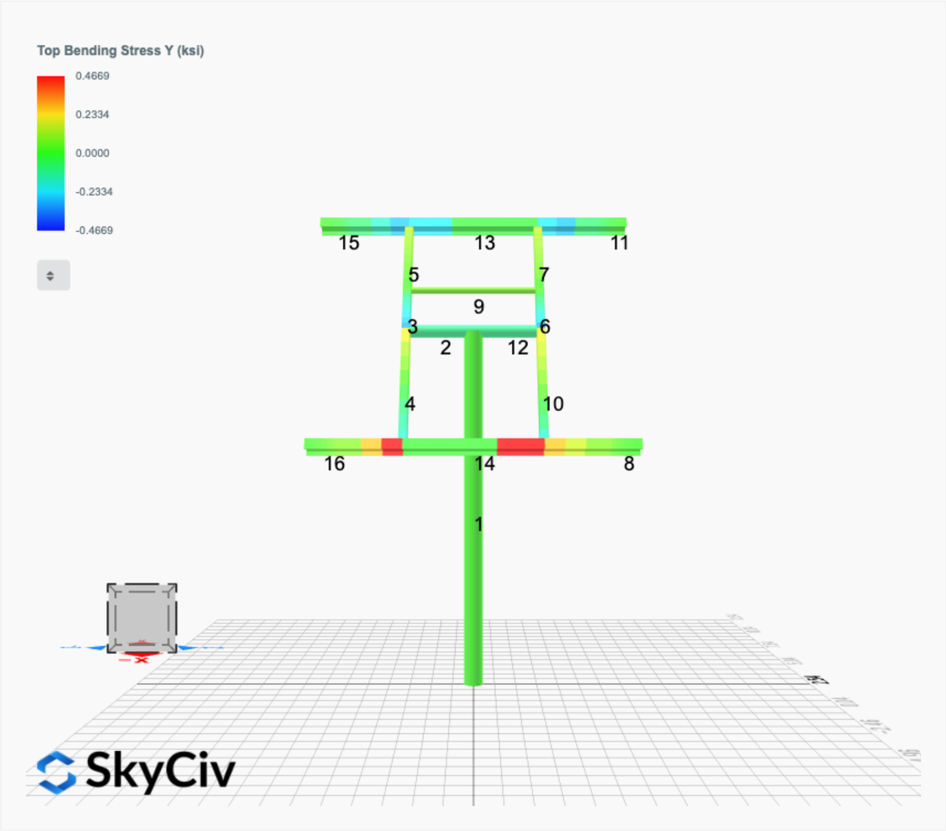
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	7.03676485	0.204	PASS
Material Yield	34.5	7.03676485	0.204	PASS
Material Strength	37	7.03676485	0.190	PASS

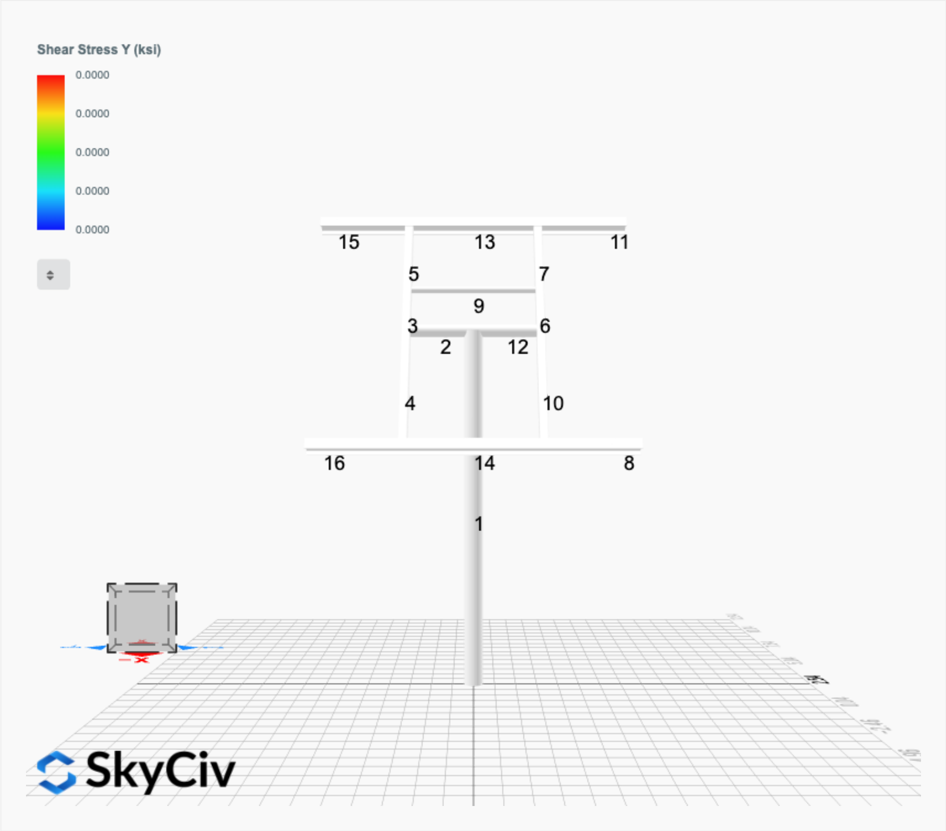


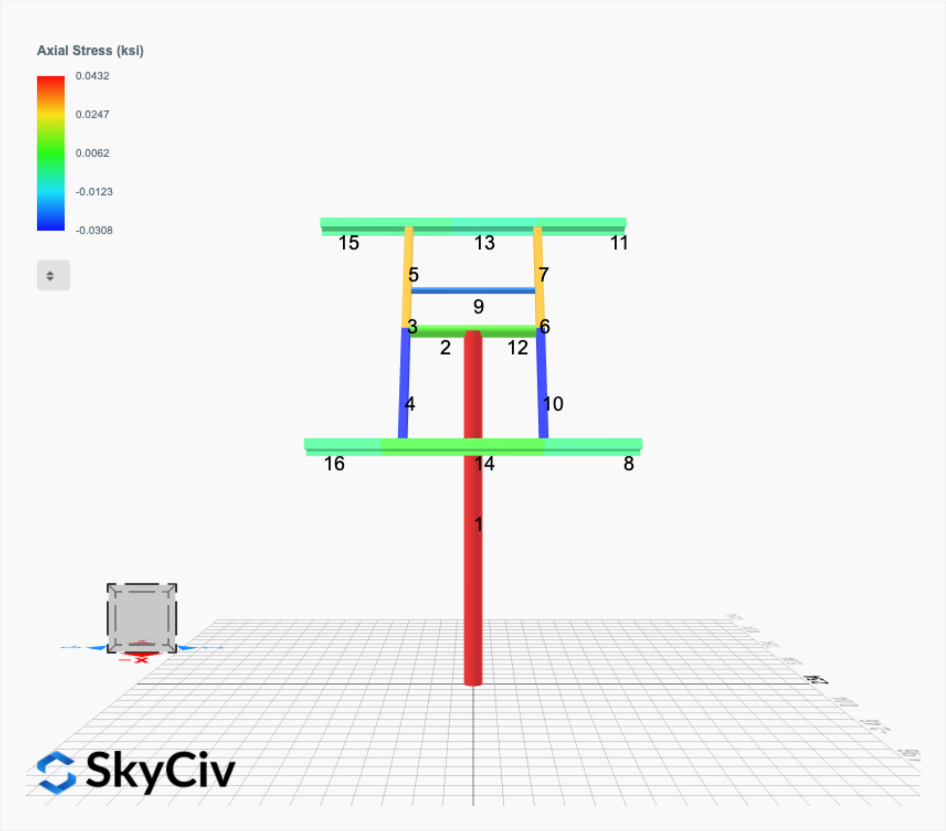
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
0.4669
0.2334
0.0000
-0.2334
-0.4669
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
0.0000
0.0000
0.0000
0.0000
0.0000
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.0432
0.0247
0.0062
-0.0123
-0.0308
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv

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	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 2. D + L	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 3. D + (S or Lr or R)	-0.0000	1.3747	0.0000	-0.0000	0.0000	0.0095
ULS: 3. D + (S or Lr or R)	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.3145	0.0000	-0.0000	0.0000	0.0094
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 5b. D + 0.7E	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.3145	0.0000	-0.0000	0.0000	0.0094
ULS: 8. 0.6D + 0.7E	-0.0000	0.6803	0.0000	-0.0000	0.0000	0.0056
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8982	2.0190	0.0000	-0.0000	0.0000	20.1229
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8982	0.2487	0.0000	-0.0000	0.0000	-19.7233
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4237	1.9784	0.0000	-0.0000	0.0000	15.0946
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.3145	0.0000	-0.0000	0.0000	0.0094
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4237	0.6506	0.0000	-0.0000	0.0000	-14.7901
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.3145	0.0000	-0.0000	0.0000	0.0094
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4237	1.7977	0.0000	-0.0000	0.0000	15.0945
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4237	0.4700	0.0000	-0.0000	0.0000	-14.7902
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.1338	0.0000	-0.0000	0.0000	0.0094
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8982	1.5655	0.0000	-0.0000	0.0000	20.1191
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.6803	0.0000	-0.0000	0.0000	0.0056
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8982	-0.2049	0.0000	-0.0000	0.0000	-19.7271
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.6803	0.0000	-0.0000	0.0000	0.0056

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	2.9563
Shear X	-3.1637
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	33.8789

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	2.0190
Shear X	-1.8982
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	20.1229

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : MTSOLAR_77LC0HCD9B1J
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Wind Loads are based on Freestanding Wall. Wind loads are applied by summing the total individual point loads then taking **worst case scenario between Case A and Case C**. We then divide this total force by the length of the members and apply as a distributed load.
Note: Case C is combined into a single load, then applied as a uniform distributed load.

Project Data

The structure is located in **637 Rainbow, Tabernash, CO 80478** categorized as **Exposure B** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Solid freestanding walls and attached signs - is based on the Directional Procedure (Chapter 29) of ASCE 7. Morever, the structure is classified as **Risk Category I**. The location is elevated at **8956 ft** above mean sea level.

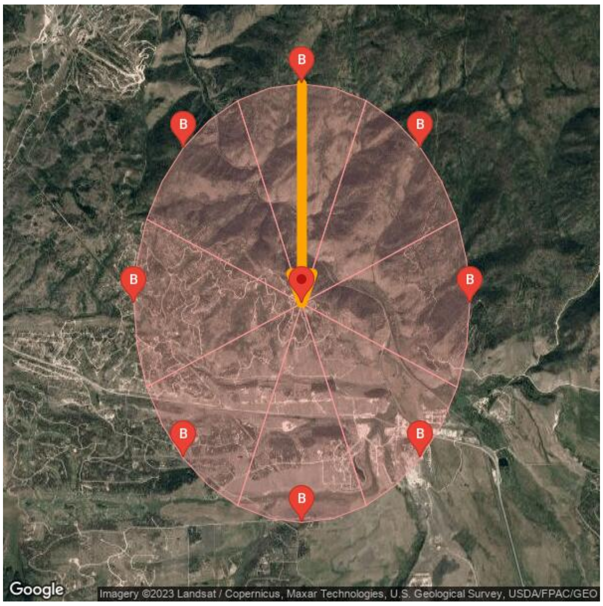


Figure 1. Site location.

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

Parameter	Value
Ground to Top of Wall/Sign, h	15.932 ft
Wall/Sign Horizontal Dimension, B	10.167 ft
Wall/Sign Vertical Dimension, s	10.876 ft
Ratio of Solid Area to Gross Area, Ɛ	1.000
Length of return corner, L_r	- ft

Figure 2. Solid Signs parameters.

Figure 26.5-1

Basic Wind Speed, V

Wind speed for the address is **115 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.

$V = 115$ mph **(defined by the user)**

Figure 26.8-1

Topographic Effects, K_{zt}

The topography factor, K_{zt}, have been calculated based on the **wind coming from N**. K_{zt} was calculated using the following formulas:

$$K_{zt} = (1 + K_1K_2K_3)^2$$

$$K_2 = (1 - |x|/\mu L_h)$$

$$K_3 = e^{-\gamma z/L_h}$$

and K_1 - determined from Figure 26.8-1

Since the topography is classified as **Hill**, topography effects should be considered. From Section 26.8.1, K_{zt} is calculated (greater than 1.0) if the location satisfies all of the following conditions:

If $H/L_h > 0.2$

$$H/L_h = 825.9658613230004/1017.540 = 0.812$$

If $H/L_h > 0.5$:

$$H/L_h = 0.5$$

$$L_h = 2H = 2(825.966) = 1651.932$$

Using Figure 26.8-1:

$$K_1 = 0.475$$

$$\gamma = 4.000$$

$$\mu = 1.500$$

$$K_2 = (1 - |528|/1.500(1651.9317226460007)) = 0.787$$

Note: K_3 varies with height:

$$K_3 = e^{-(4.000)z/(1651.932)}$$

K_{zt} varies with height

Table 26.6-1

Wind Directionality Factor, K_d

The wind directionality factors, K_d, for the structure is equal to **0.85** (for MWFRS, and Components and Claddings) based on Table 26.6-1.

$K_d = 0.85$

Section 26.9.1

Gust Effect Factor, G

The structure is assumed to be rigid, hence, gust effect factor, G, is set to **0.85** based on Section 26.9.1.

$G = 0.85$

Table 26.9-1

Groud Elevation Factor, K_e

The location is elevated at 8955.5 ft above mean sea level. To account for air density, K_e is calculated in accordance with Table 26.9-1 using the formula:

$$K_e = e^{-0.0000362z_g}$$

$$K_e = e^{-0.0000362(8955.5)} = 0.723$$

$K_e = 0.723$

Section 26.10
Table 26.10-1

Velocity Pressure Exposure Coefficient, K_z and Velocity Pressure, q_z

The velocity pressures, q_z, shall be computed using the equation:

$$q_z = 0.00256K_zK_{zt}K_dK_eV^2$$

$$q_z = 0.00256K_zK_{zt}(0.85)(0.723)(115)^2$$

where: K_z and K_{zt} is calculated for each height using Table 27.3-1 rounded to nearest hundredth. The table below shows the **comparison of calculated q_z values for each parameter depending on the Exposure Category of each wind source direction to generate the worst case wind direction:**

Wind Direction	Exposure Category	Velocity Pressure Exposure Coefficient K_z @ 15.932 ft	Topographic factor K_{zt}@ z = 0 ft	Wind Directionality factor K_d	Ground Elevation factor K_e	Basic Wind Speed V, mph	Velocity Pressure q_h, psf
N	B	0.580	1.000	0.850	0.723	115.000	12.069
S	B	0.580	1.000	0.850	0.723	115.000	12.069
E	B	0.580	1.000	0.850	0.723	115.000	12.069
W	B	0.580	1.000	0.850	0.723	115.000	12.069
NE	B	0.580	1.000	0.850	0.723	115.000	12.069
SE	B	0.580	1.000	0.850	0.723	115.000	12.069
NW	B	0.580	1.000	0.850	0.723	115.000	12.069
SW	B	0.580	1.000	0.850	0.723	115.000	12.069

From the formula above, the calculated K_z and q_z per level for **Wind Source Direction N - Exposure Category B** are as follows:

1.849

Level	Height, ft	K_{zt}	K_z	q_z, psf
Ground to Top of Wall/Sign	15.932	0.58	22.31	

Figure 29.3-1 of ASCE 7-16

Net Force Coefficient, C_f

The net force coefficients, C_f, for Case A and Case B are calculated using Figure 29.3-1 of ASCE 7-16. Note that the values are interpolated using known values for each s/h and B/s value:

$$B/s = 10.17/10.88 = 0.935$$

$$s/h = 10.88/15.93 = 0.683$$

Reduction Factor for signs with opening:

$$R_{factor,open} = 1 - (1 - \varepsilon)^{1.5} = 1 - (1 - 1.000)^{1.5} = 1.000$$

For Case A:

$$C_{f,A} = R_{factor,open} C_{f,A} = (1.000)(1.665) = 1.665$$

For Case B:

$$C_{f,B} = R_{factor,open} C_{f,B} = (1.000)(1.665) = 1.665$$

Design wind Force, F

Equation 29.3-1 of ASCE 7-16

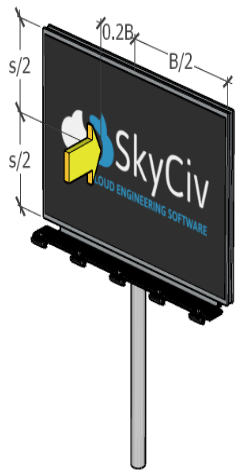
The design wind force, F, can be calculated using Equation 29.3-1 of ASCE 7-16.

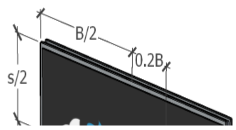
$$F = q_h G C_f A_s = (22.31)(0.85) C_f (110.57) = 2097.025 C_f$$

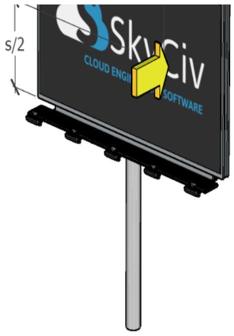
The design forces for each case is summarized on table below:

Case	Location	C_f	Design Force, F lb
Case A	e = 0 ft	1.665	3490.772
Case B	e = 2.03 ft	1.665	3490.772

Figure 3. Case A.





	 **Figures 4 and 5. Case B.**		

REFERENCES	CALCULATIONS	RESULTS

Snow Load Detailed Calculations based on ASCE 7-16

Design Information :

Project Name : MTSOLAR_77LC0HCD9B1J
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **637 Rainbow, Tabernash, CO 80478** 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 **8956 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	5.071 ft
Building Width, B	10.000 ft
Mean Roof Height, h	10.494 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	65.000°

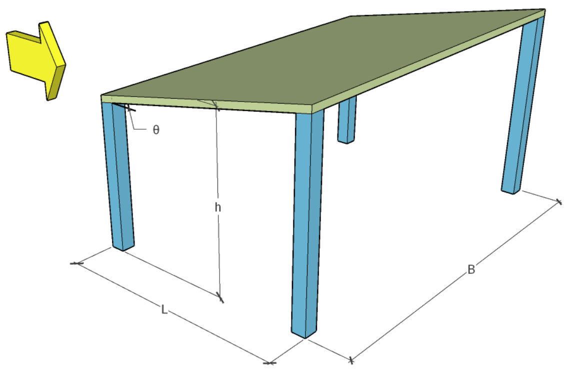


Figure 2. Building parameters.

Ground Snow Load, p_g

Section 7.2 of ASCE 7

The ground snow load, p_g, for the site location is **90 psf (defined by the user)** at elevation 8955.50 ft above mean sea level based on Section 7.2 of ASCE 7

$p_g = 90$ psf **(defined by the user)**

	r based on Section 7.2 of ASCE 7.	y 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: $$p_f = 0.7 C_e C_t I_s p_g$$ $$p_f = 0.7(0.90)(1.20)(0.80)(90.00) = 54.43 psf$$	$p_f = 54.43$ 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: $$\gamma = 0.13 p_g + 14 \leq 30 = 0.13(90.00) + 14 \leq 30$$ $$\gamma = 25.70 pcf$$	$\gamma = 25.70 pcf$
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 0.091 based on Figure 7.2c where $\theta = 65.00°$.	$C_s = 0.091$
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: $$p_s = C_s p_f$$ $$p_s = (0.091)(54.43) = 4.95 psf$$	$p_s = 4.95$ 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

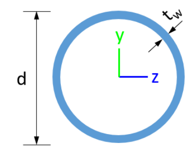
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

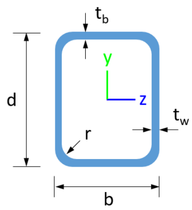
Design Materials

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

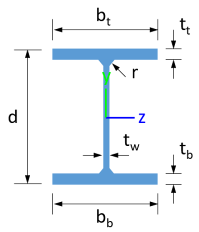
Section Dimensions



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	22.04	22.04	10.49	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
4	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
8	18	2.10	2.10	1.00	2.32,2.32,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
11	18	2.10	2.10	1.00	2.32,2.32,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	300	200	1
14	18	4.88	4.00	7.50	1.15,1.15,1.15,1.15,1.15,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15	300	200	1
15	18	2.10	2.10	1.00	2.32,2.32,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
16	18	2.10	2.10	1.00	2.32,2.32,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1

Member Design Capacity

Member ID	Φ_tP_n (kip)	Φ_cP_n (kip)	Φ_bM_{zn} (k-ft)	Φ_bM_{yn} (k-ft)	Φ_vV_{yn} (kip)	Φ_vV_{zn} (kip)
1	251.16	90.90	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	113.97	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	113.97	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	20.42	6.45	30.09	45.74
14	120.60	98.23	20.25	6.45	30.09	45.74
15	120.60	113.97	23.36	6.45	30.09	45.74
16	120.60	113.97	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	0.033	0.801	0.000	0.042	0.000	0.817	#13	0.589	Not Required	Pass
2	0.001	0.132	0.179	0.032	0.037	0.311	#13	0.052	Not Required	Pass
3	0.005	0.342	0.065	0.034	0.013	0.397	#13	0.044	Not Required	Pass
4	0.004	0.341	0.039	0.034	0.005	0.374	#13	0.078	Not Required	Pass
5	0.004	0.212	0.026	0.034	0.003	0.216	#13	0.073	Not Required	Pass
6	0.005	0.342	0.065	0.034	0.013	0.397	#13	0.044	Not Required	Pass
7	0.004	0.212	0.026	0.034	0.003	0.216	#13	0.073	Not Required	Pass
8	0.000	0.004	0.005	0.007	0.001	0.008	#13	Not Required	Not Required	Pass
9	0.004	0.013	0.028	0.001	0.000	0.043	#13	0.132	Not Required	Pass
10	0.004	0.341	0.039	0.034	0.005	0.374	#13	0.078	Not Required	Pass
11	0.000	0.004	0.005	0.007	0.001	0.008	#13	Not Required	Not Required	Pass
12	0.001	0.132	0.179	0.032	0.037	0.311	#13	0.052	Not Required	Pass
13	0.000	0.047	0.037	0.019	0.004	0.068	#13	0.177	Not Required	Pass
14	0.001	0.049	0.037	0.019	0.004	0.068	#13	0.177	Not Required	Pass
15	0.000	0.004	0.005	0.007	0.001	0.008	#13	Not Required	Not Required	Pass
16	0.000	0.004	0.005	0.007	0.001	0.008	#13	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

NC	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

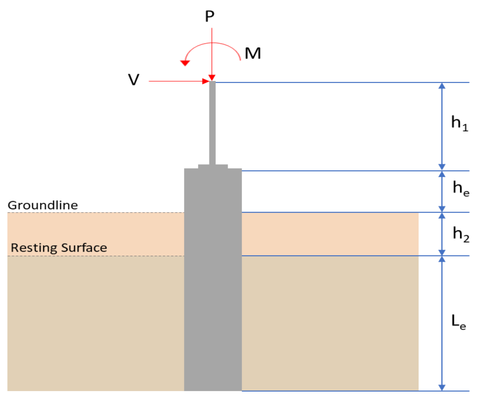
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 8.25\ \mathrm{ft}$ - 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) (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)	2.019	2.956
V_x (kip)	-1.898	-3.164
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	20.123	33.879

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - 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,

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

$$H_o = \frac{(-1.898\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.63267\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.123 \text{ kipft}) + ((-1.898 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.7077 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.3838 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(7.3838 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.384 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.384 \text{ ft})}{(8.25 \text{ ft})}$$

$$Ratio = 0.89503$$

Status: **PASS**
Ratio: **0.900**

End-bearing Capacity (ASD)

A - Pile cros-section area

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

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(2.019 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.28563 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.28563 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.14282$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.75$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.63267\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.7077\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7077\ \mathrm{kipft/ft}) \times (8.25\ \mathrm{ft})) + \left(3 \times (-0.63267\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft})^2\right)}{(6 \times (6.7077\ \mathrm{kipft/ft})) + (4 \times (-0.63267\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))}$$

$$a = 5.7348\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (6.7077\ \mathrm{kipft/ft})) + (3 \times (-0.63267\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))]^2}{(8.25\ \mathrm{ft})^2 \times [(3 \times (6.7077\ \mathrm{kipft/ft})) + (2 \times (-0.63267\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))]}$$

$$p = 0.22308\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (6.7077\ \mathrm{kipft/ft})) + ((-0.63267\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))]}{(8.25\ \mathrm{ft})^2}$$

$$s = 1.1349\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.7348\ \mathrm{ft})}{2}$$

$$p_a = 0.43011\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22308\ \mathrm{kip/ft^2}\right)}{\left(0.43011\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.51865$$

Status: **PASS**
Ratio: **0.520**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8.25\ \mathrm{ft})$$

$$p_s = 1.2375\ \mathrm{kip/ft^2}$$

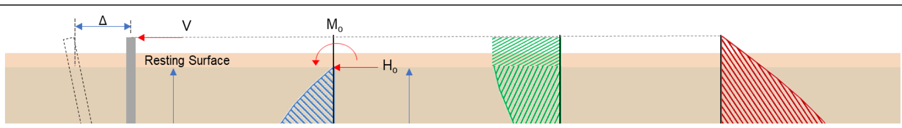
$Ratio$ - Lateral soil capacity

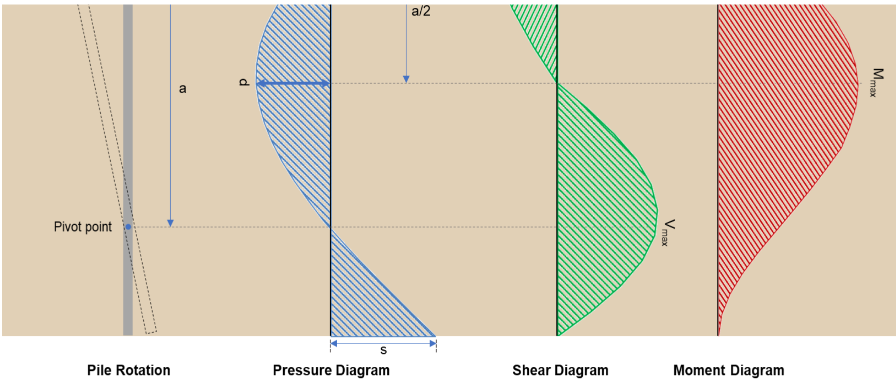
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.1349\ \mathrm{kip/ft^2}\right)}{\left(1.2375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.91711$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.164\ \text{kip})}{(36\ \text{in})}$$

$$H_o = -1.0547\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.879\ \text{kipft}) + ((-3.164\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 11.293\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.293\ \text{kipft/ft})}{(-1.0547\ \text{kip/ft})}$$

$$E = 10.708\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.293\ \text{kipft/ft}) \times (8.25\ \text{ft})) + \left(3 \times (-1.0547\ \text{kip/ft}) \times (8.25\ \text{ft})^2\right)}{(6 \times (11.293\ \text{kipft/ft})) + (4 \times (-1.0547\ \text{kip/ft}) \times (8.25\ \text{ft}))}$$

$$a = 5.7333\ \text{ft}$$

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

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

$$V_{max} = ((-1.0547\ \text{kip/ft}) \times (36\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.708\ \text{ft})}{(8.25\ \text{ft})} + 3\right) \times \left(\frac{(5.7333\ \text{ft})}{(8.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.708\ \text{ft})}{(8.25\ \text{ft})} + 2\right) \times \left(\frac{(5.7333\ \text{ft})}{(8.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.3532\ \text{kip}$$

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

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

$$M_{max} = ((-1.0547\ \text{kip/ft}) \times (36\ \text{in}) \times (8.25\ \text{ft})) \times \left[\left(\frac{(10.708\ \text{ft})}{(8.25\ \text{ft})} + \frac{(5.7333\ \text{ft})}{2 \times (8.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (10.708\ \text{ft})}{(8.25\ \text{ft})} + 3\right) \times \left(\frac{(5.7333\ \text{ft})}{2 \times (8.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.708\ \text{ft})}{(8.25\ \text{ft})} + 2\right) \times \left(\frac{(5.7333\ \text{ft})}{2 \times (8.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 36.221 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(2.956 \text{ kip})}{(0.65)\times(0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2))\right]$$

$$A_{st,required} = -45.087 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-45.087 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))\right]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

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,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

	Main reinforcement: **6 - #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 $$\phi P_N = \phi\, 0.85\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 1492.5\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(2.956\text{ kip})}{(1492.5\text{ kip})}$$ $$Ratio = 0.0019806$$	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:** $b_w = 36\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (36\text{ in})$$ $$d = 28.8\text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.71796$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,max} = 203.86\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $P = 2.956\text{ kip} \rightarrow 2956\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + \frac{(2956\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,a} = 82.044\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,b} = 237.06\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(203.86\text{ kip}), (82.044\text{ kip}), (237.06\text{ kip})\right]$$	

$$V_c = 82.044 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

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

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((82.044 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 78.14 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.3532 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.3532 \text{ kip})}{(78.14 \text{ kip})}$$

$$Ratio = 0.1197$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 4580.442 in^3$$

$$\phi M_{n,1} = 67.947 kipft$$

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

	$$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3\ \mathrm{ksi}) \times \left(4580.4\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 632.67\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(67.947\ \mathrm{kipft}), (632.67\ \mathrm{kipft})]$$ $$\phi M_n = 67.947\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 36.221\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(36.221\ \mathrm{kipft})}{(67.947\ \mathrm{kipft})}$$ $$Ratio = 0.53308$$		Status: **PASS** Ratio: **0.530**