Project Details

Project Name: Wayman-v2cu-sq - revised mod size

Location: XCMQ+2J2, Animas Forks, CO 81433, USA

Unique ID: 1P-0-6TOP-HD-24-L-5Hx2W-K51A

Dealer: _____

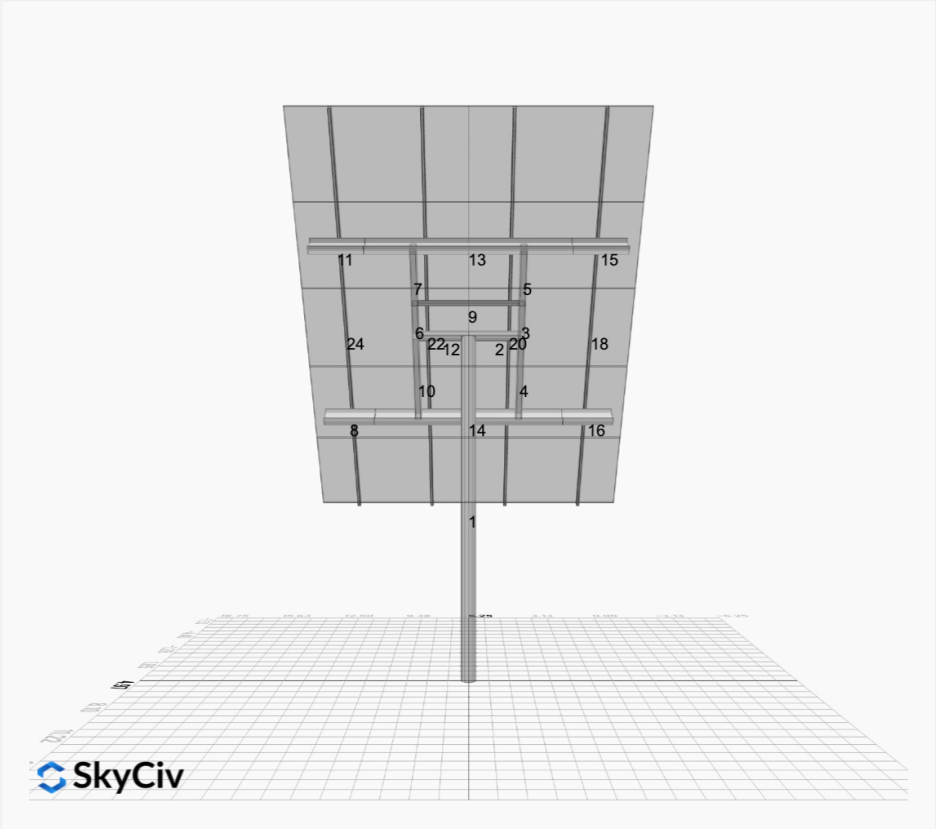
Date: Tue Aug 26 2025

Number of Modules: 10

Number of Poles: 1

Date Sold:

_____



Array Dimensions N/S	17.33 ft
Array Dimensions E/W	12.50 ft
Winter Tilt Angle	55
Front Edge Clearance	6 ft

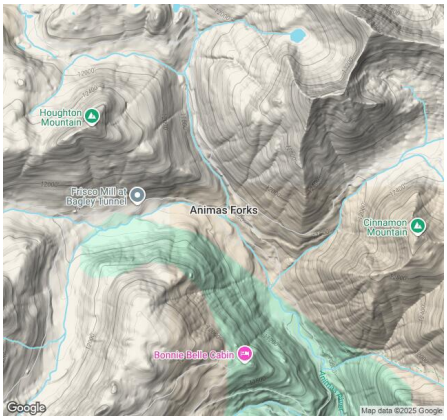
MT Solar Bill of Materials (1P-0-6TOP-HD-24-L-5Hx2W-K51A)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-24	24IN HD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	2

Rail Bill of Materials

Part	Qty
Rails (208in)	4
Rail Attachment	16
Module Mid Clamp	16
Module End Clamp	8
Ground Lug	2

Site Details:



Site Address: XCMQ+2J2, Animas Forks, CO 81433, USA

Array Specification

Duty Classification:	HD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	5
Number of Columns:	2
Total Number of Modules:	10
Winter Tilt Angle:	55
Front Edge Clearance:	6
Total Array Height at Tilt:	20.20 ft
Total Frame Length:	11.50 ft
Module Info/Notes:	
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	12.50 ft
Rail Length:	208.00 in
Rail Spacing:	3.13 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	13.10 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.50 ft
Foundation Volume:	2.667 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sedimentary
Site Location:	XCMQ+2J2, Animas Forks, CO 81433, USA
Wind Speed:	100 mph
Snow Load:	321 psf

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


```
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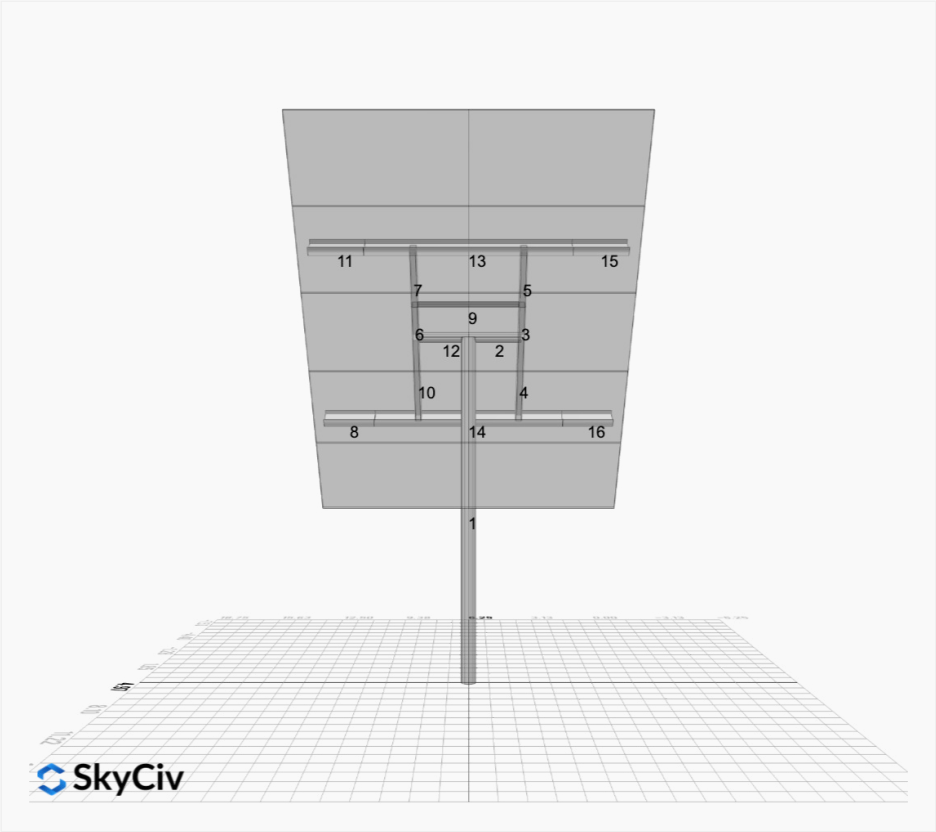
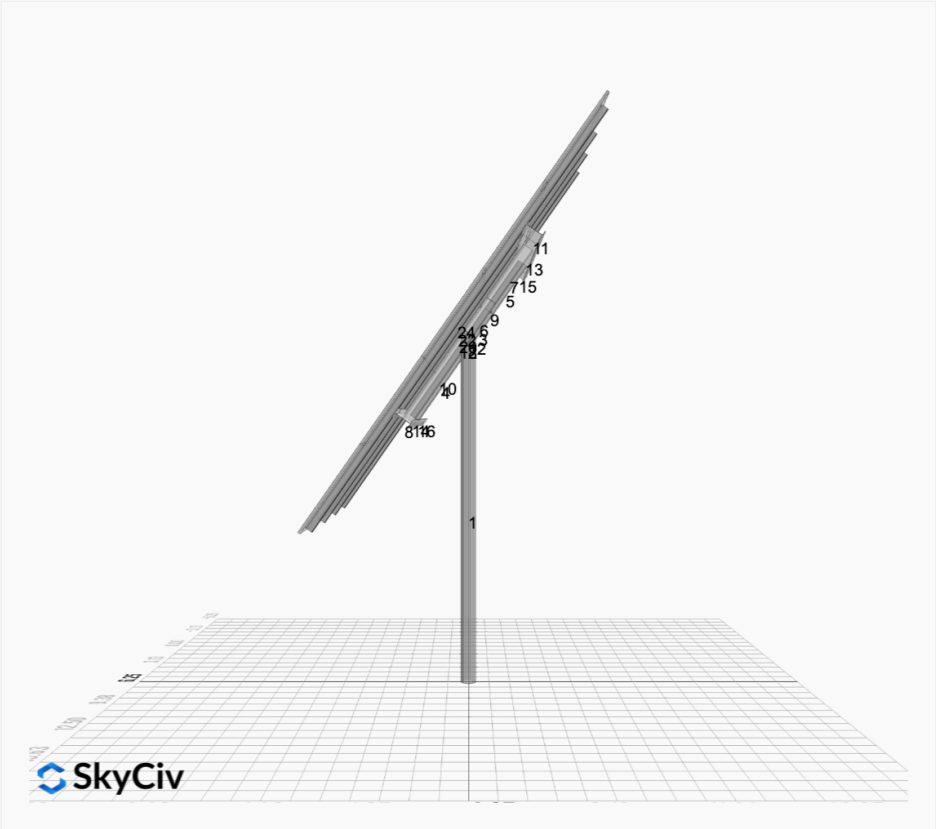
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)

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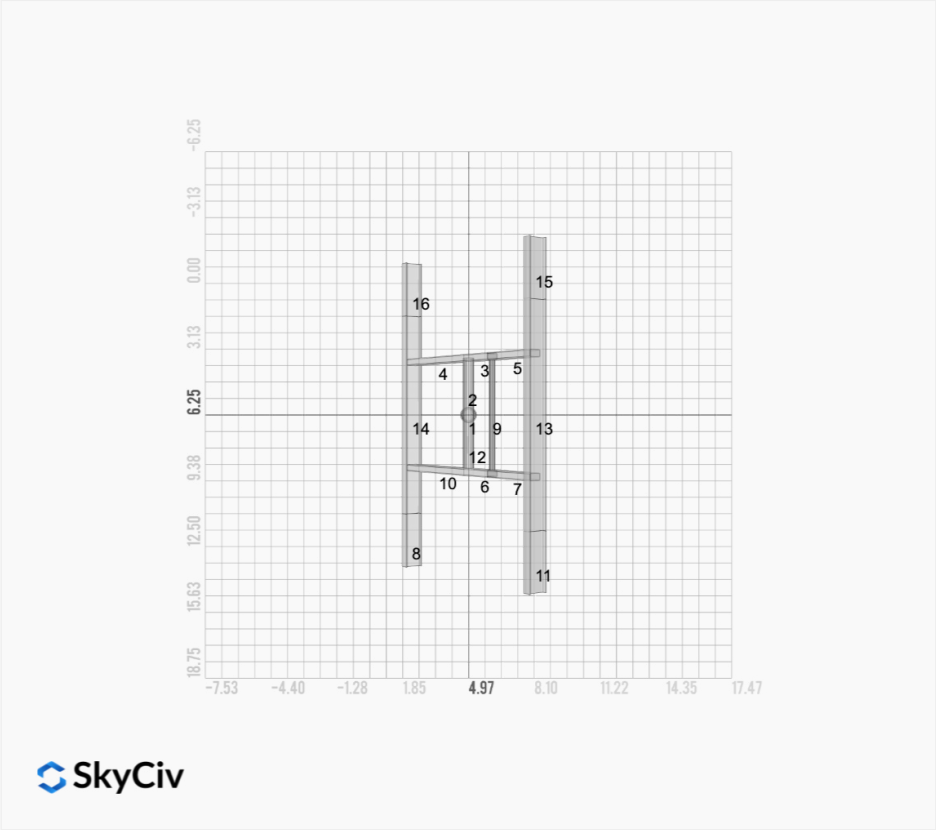
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3.13
6.25
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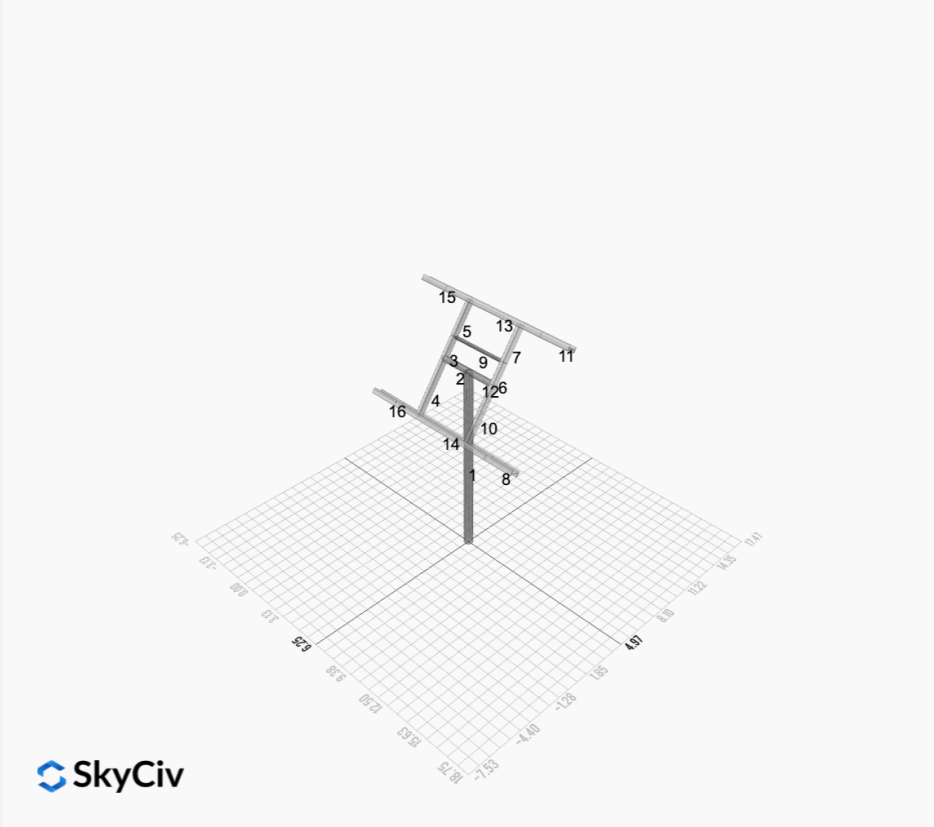
1.85
4.97
8.10
11.22
14.35
17.47
SkyCiv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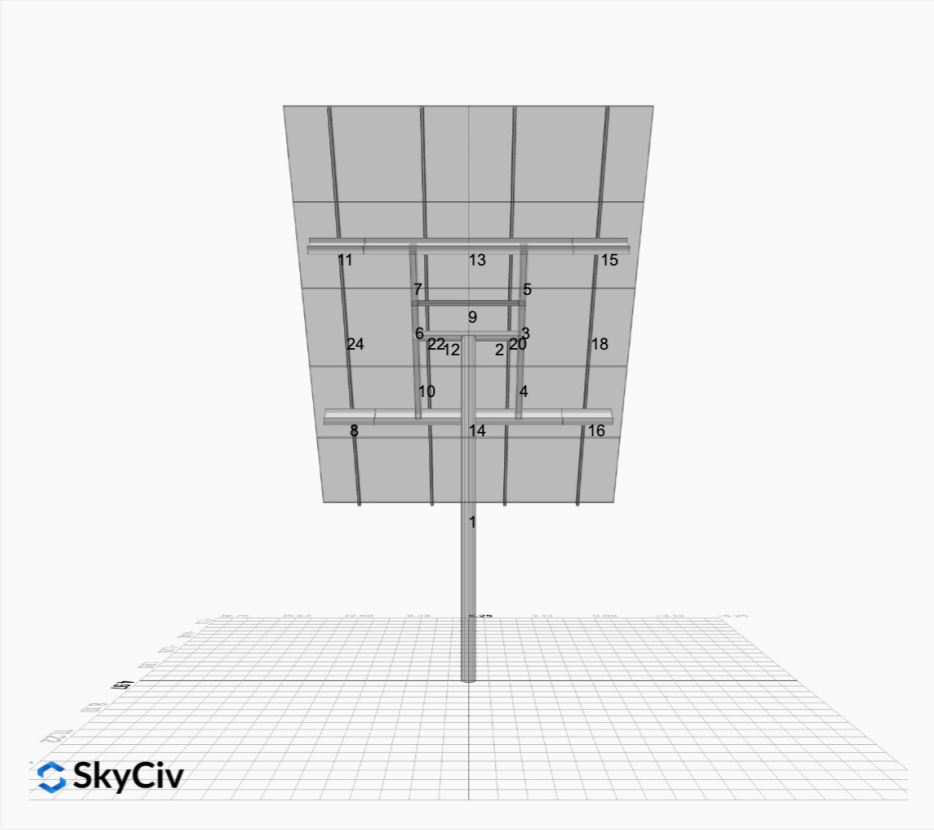
8
SkyCiv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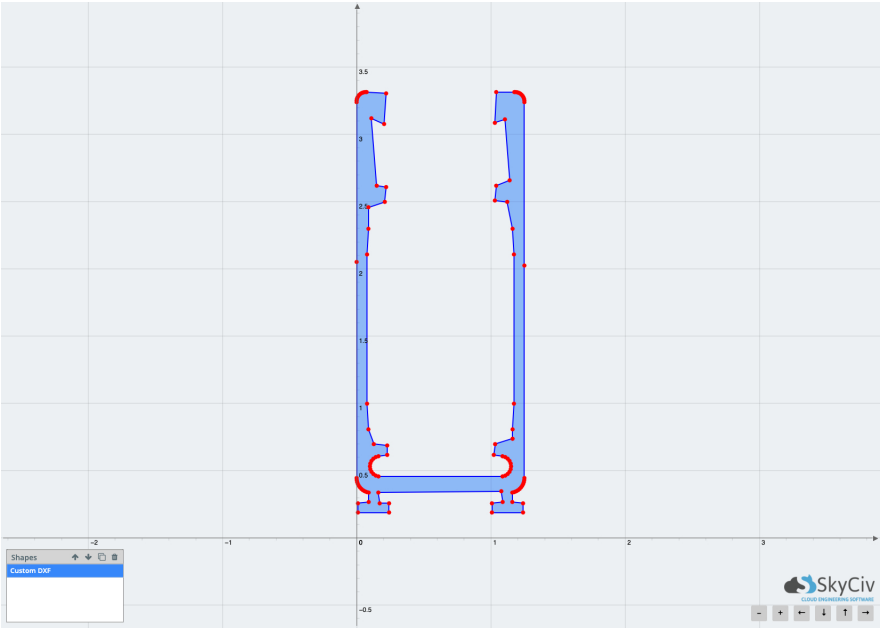
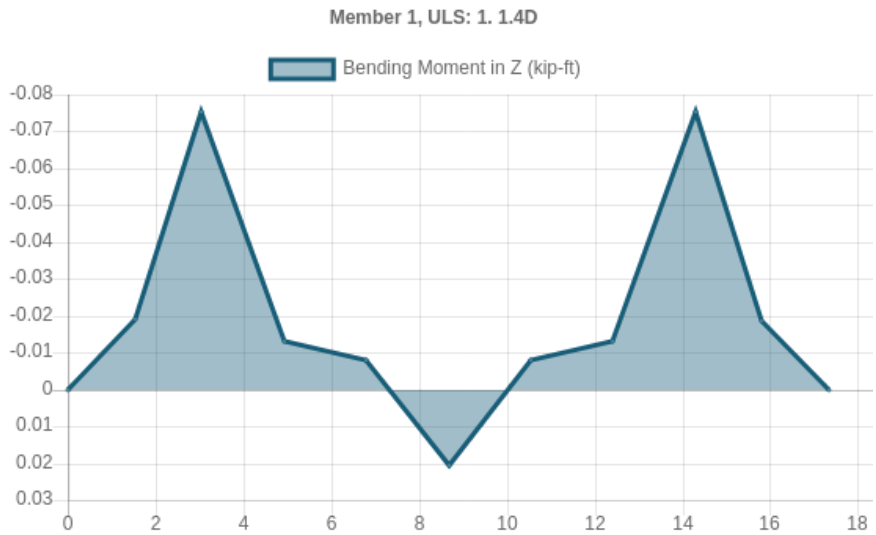
13
15
7
5
9
6
3
24
22
12
2
20
18
10
4
8
14
16
1
SkyCiv

Rail Design Check

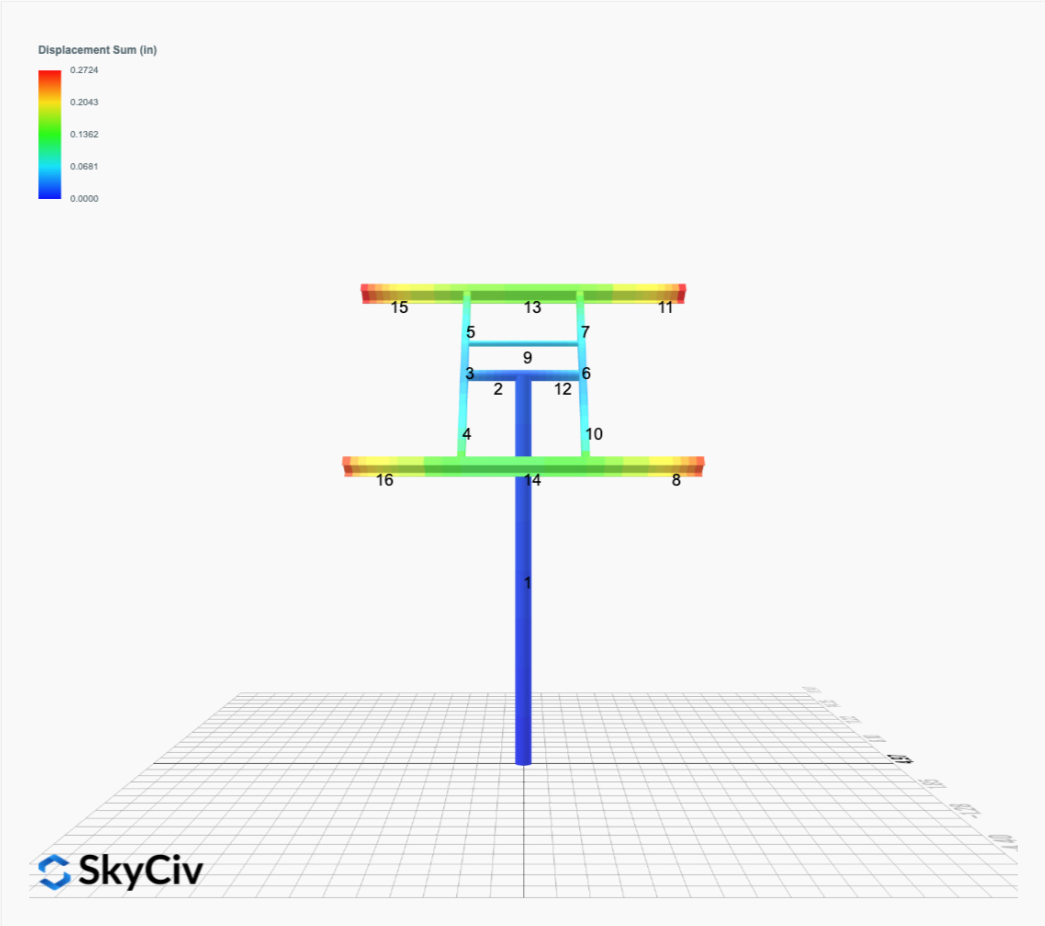
Rail Length: 17.333333333333332 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.125 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0949 kip/ft
Snow (Y): -0.1355 kip/ft
Wind uplift Case A: 0.0466 kip/ft
Wind downforce Case A: 0.0466 kip/ft
Dead (Panel load) (X): 0.0082 kip/ft
Dead (Panel load) (Y): -0.0117 kip/ft

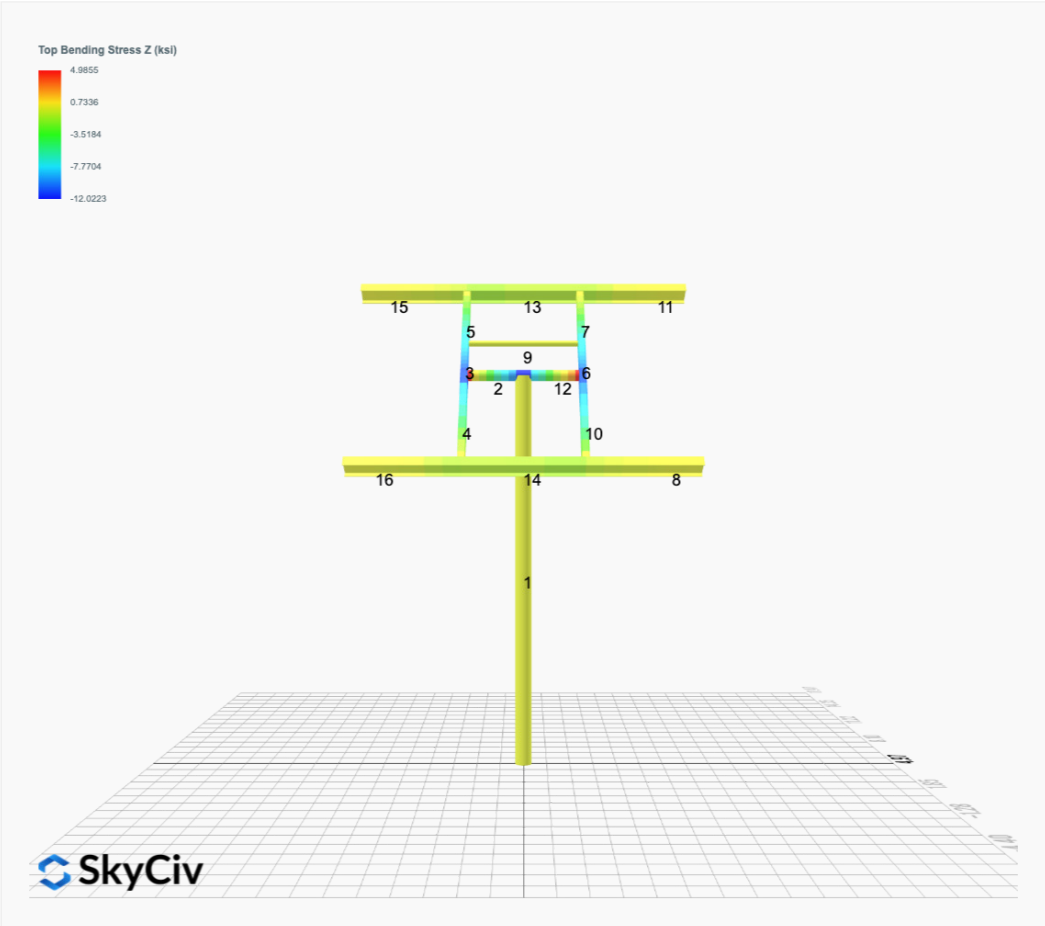


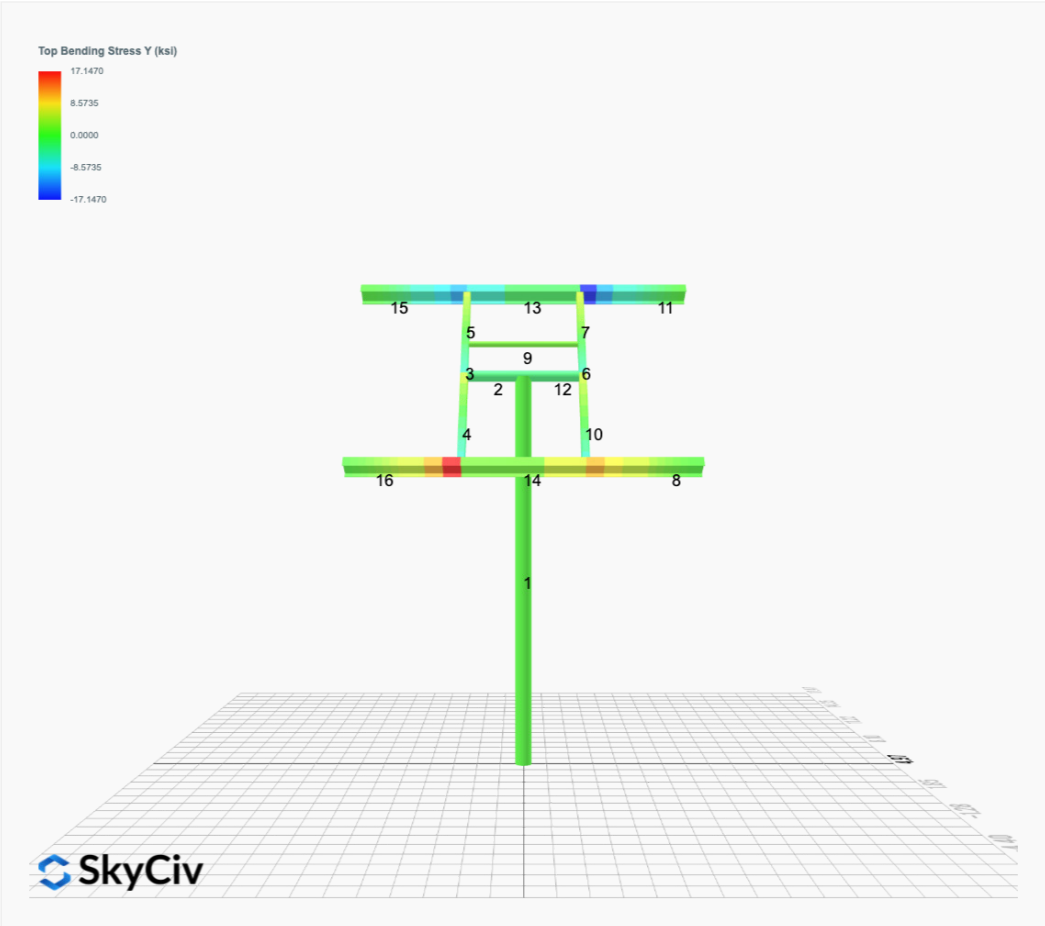
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	25.72915508	0.746	PASS
Material Yield	34.5	25.72915508	0.746	PASS
Material Strength	37	25.72915508	0.695	PASS



FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
17.1470
8.5735
0.0000
-8.5735
-17.1470
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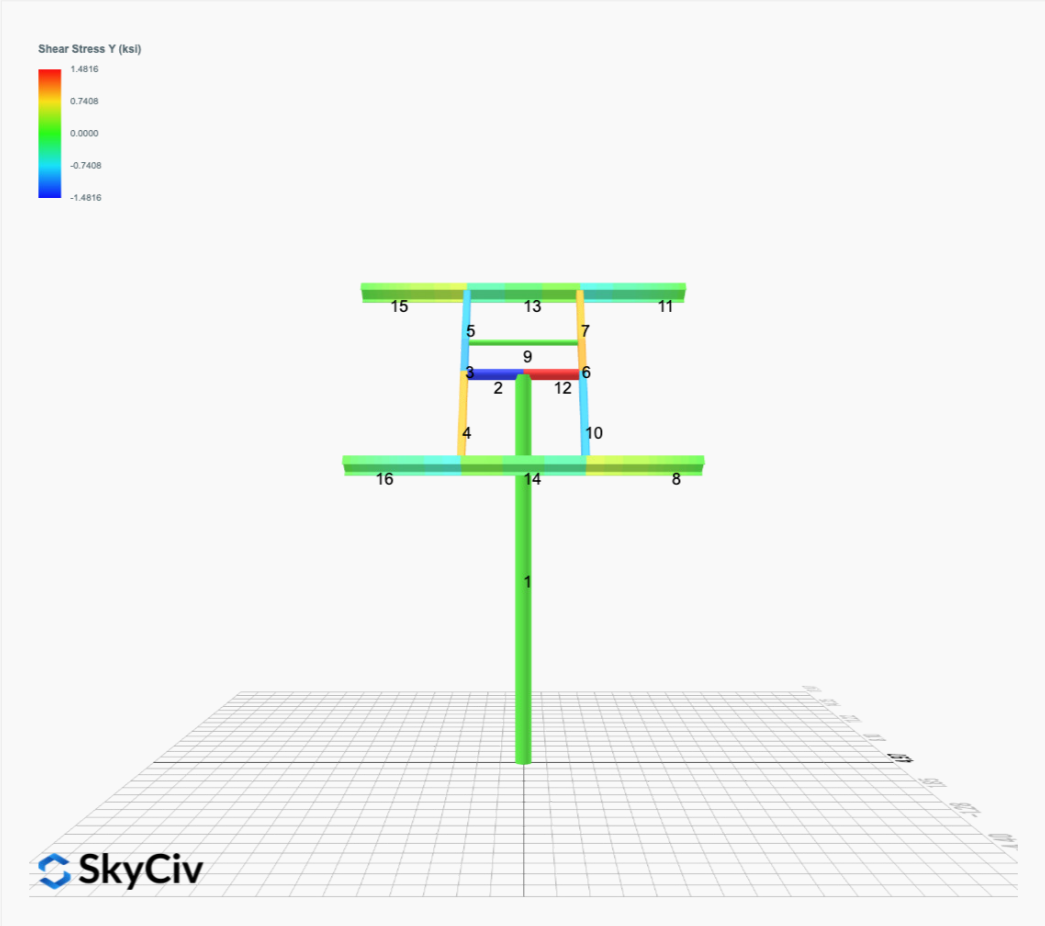
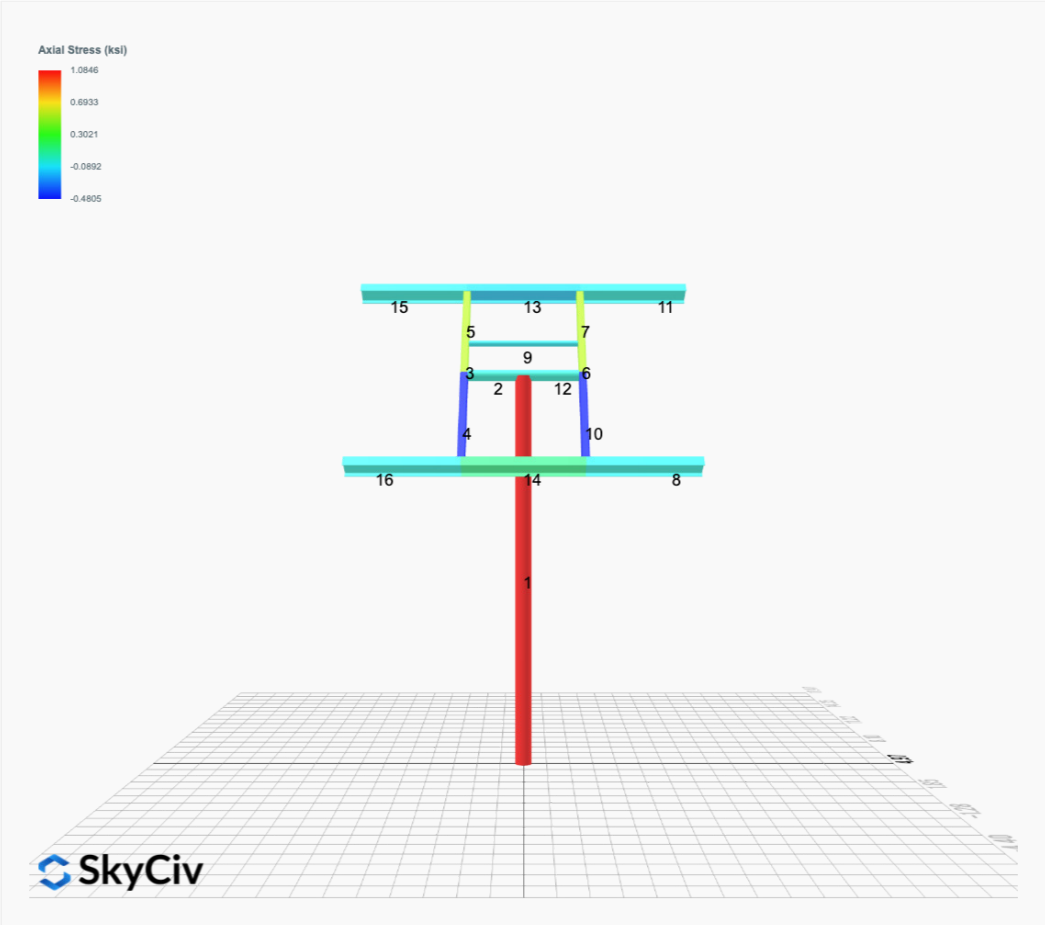
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
1.4816
0.7408
0.0000
-0.7408
-1.4816
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
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0.6933
0.3021
-0.0892
-0.4805
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	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 2. D + L	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 3. D + (S or Lr or R)	0.0000	7.7325	0.0000	0.0000	-0.0000	0.0662
ULS: 3. D + (S or Lr or R)	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.2191	0.0000	0.0000	-0.0000	0.0541
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 5b. D + 0.7E	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.2191	0.0000	0.0000	-0.0000	0.0541
ULS: 8. 0.6D + 0.7E	0.0000	1.0073	0.0000	0.0000	-0.0000	0.0107
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5876	2.7906	0.0000	0.0000	-0.0000	21.2124
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5876	0.5672	0.0000	0.0000	-0.0000	-20.3987
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1907	7.0529	0.0000	0.0000	-0.0000	15.9501
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	6.2191	0.0000	0.0000	-0.0000	0.0541
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1907	5.3854	0.0000	0.0000	-0.0000	-15.2583
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	6.2191	0.0000	0.0000	-0.0000	0.0541
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1907	2.5126	0.0000	0.0000	-0.0000	15.9138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1907	0.8451	0.0000	0.0000	-0.0000	-15.2946
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0178
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5876	2.1190	0.0000	0.0000	-0.0000	21.2053
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0073	0.0000	0.0000	-0.0000	0.0107
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5876	-0.1043	0.0000	0.0000	-0.0000	-20.4058
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0073	0.0000	0.0000	-0.0000	0.0107

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	12.6269
Shear X	-2.6460
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	37.3801

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	7.7325
Shear X	-1.5876
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	21.2124

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : Wayman-v2cu-sq - revised mod size
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 **XCMQ+2J2, Animas Forks, CO 81433, USA** categorized as **Exposure C** (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. Moreover, the structure is classified as **Risk Category I**. The location is elevated at **11544 ft** above mean sea level.

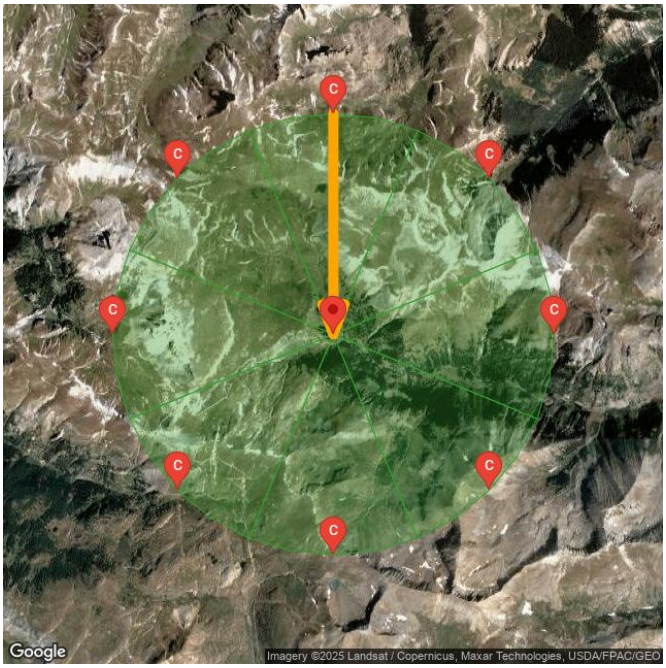


Figure 1. Site location.

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

Parameter	Value
Ground to Top of Wall/Sign, h	20.199 ft
Wall/Sign Horizontal Dimension, B	12.500 ft
Wall/Sign Vertical Dimension, s	14.199 ft
Ratio of Solid Area to Gross Area, ε	1.000
Length of return corner, L_r	0.000 ft

Figure 2. Solid Signs parameters.

Figure 26.5-1

Basic Wind Speed, V

Wind speed for the address is **100 mph (defined by the user)** for Risk Category I and was calculated using Triangular Interpolation Net work (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.

$V = 100$ 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:

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

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

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

and K_1 - determined from Figure 26.8-1

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

If $H/L_h > 0.2$

$$H/L_h = 1377.329588562172/5827.120 = 0.236$$

Using Figure 26.8-1:

$$K_1 = 0.201$$

$$\gamma = 2.500$$

$$\mu = 1.500$$

$$K_2 = (1 - |-10348.8|/1.500(5827.119999999999)) = 0.000$$

Note: K_3 varies with height:

$$K_3 = e^{-(2.500)z/(5827.120)}$$

Hence:

$$K_{zt} = (1 + (0.201)(0.000)K_3)^2 = 1.0$$

$K_{zt} = 1.0$

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 11544.15 ft above mean sea level. To account for air density, K_e is calculated in accordance with Table 26.9-1 using the formula:

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

$$K_e = e^{-0.0000362(11544.15)} = 0.658$$

$K_e = 0.658$

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:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$$

$$q_z = 0.00256 K_z (1)(0.85)(0.658)(100)^2$$

where: K_z is calculated for each height using Table 27.3-1 rounded to nearest hundredth. The table below shows the **comparison of calc ulated q_z values for each parameter depending on the Exposure Category of each wind source direction to generate the w orst case wind direction:**

Wind Dire ction	Exposure Ca tegory	Velocity Pressure Exposur e Coefficient K_z @ 20.199 ft	Topographic factor K_{zt}@ z = 0 ft	Wind Directionali ty factor K_d	Ground Elevatio n factor K_e	Basic Wind Speed V, mph	Velocity Pre ssure q_h, psf
N	C	0.900	1.000	0.850	0.658	100.000	12.895
S	C	0.900	1.000	0.850	0.658	100.000	12.895
E	C	0.900	1.000	0.850	0.658	100.000	12.895
W	C	0.900	1.000	0.850	0.658	100.000	12.895
NE	C	0.900	1.000	0.850	0.658	100.000	12.895
SE	C	0.900	1.000	0.850	0.658	100.000	12.895
NW	C	0.900	1.000	0.850	0.658	100.000	12.895
SW	C	0.900	1.000	0.850	0.658	100.000	12.895

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

1.000

Level	Height, ft	K_z	q_z, psf
Ground to Top of Wa ll/Sign	20.199	0.9	12.89

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 interpolate d using known values for each s/h and B/s value:

$$B/s = 12.50/14.20 = 0.880$$

$$s/h = 14.20/20.20 = 0.703$$

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:

$$C_{f,A} = R_{factor,open} C_{f,A} = (1.000)(1.660) = 1.660$$

For Case B:

$$C_{f,B} = R_{factor,open} C_{f,B} = (1.000)(1.660) = 1.660$$

Equation 29.3-1 of ASCE 7-16

Design wind Force, F

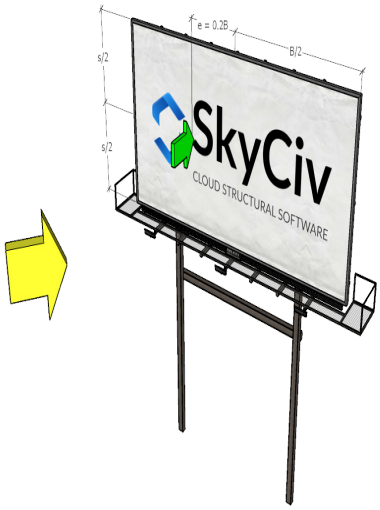
The design wind force, F, can be calculated using Equation 29.3-1 of ASCE 7-16.

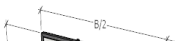
$$F = q_h G C_f A_s = (12.89)(0.85) C_f (177.49) = 1945.348 C_f$$

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

Case	Location	C_f	Design Force, F lb
Case A	e = 0 ft	1.660	3230.226
Case B	e = 2.50 ft	1.660	3230.226

Figure 3. Case A.





Figures 4 and 5. Case B.

REFERENCES	CALCULATIONS	RESULTS

Snow Load Calculations based on ASCE 7-16

Design Information :

Project Name : Wayman-v2cu-sq - revised mod size
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **XCMQ+2J2, Animas Forks, CO 81433, USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on t he Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **11544 ft** above mean sea level.

Figure 1. Site location.

Parameter	Value
Building Length, L	9.942 ft
Building Width, B	12.500 ft
Mean Roof Height, h	13.099 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	55.000°

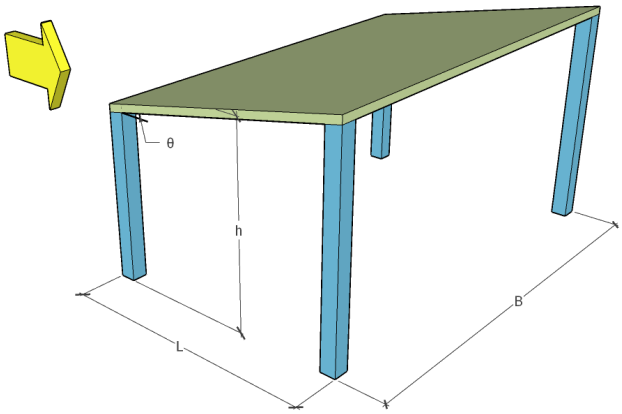


Figure 2. Building parameters.

Figure 7.6-2 S

Where roof slope is equal to $\frac{1}{S}$

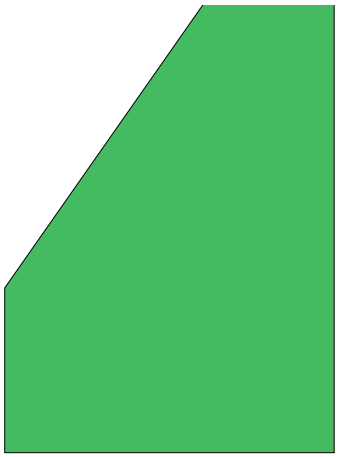
$$S = \frac{1}{tan\theta}$$

$$S = 0.70021$$

$$\text{Where } \theta = \text{Angle of slope of roof} = 55\ ^{\circ}$$

Ground Snow Load. p_o

From Fig. 7.2-1 and Table 7.2-1 ATC/ASCE 7 Hazard Tools *Google Maps*	$p_g = 321$ psf - Ground Snow Load *User-defined value* $E = 11544$ ft - Ground Elevation above mean sea level	$p_g = 321$ psf $E = 11544$ ft
Table 7.3-1 of ASCE 7-16	**Exposure Factor,** C_e $C_e = 0.9$ - Exposure Factor *For Terrain Category **Surface Roughness C** with exposure condition specified as Fully Exposed.*	$C_e = 0.9$
Table 7.3-2 of ASCE 7-16	**Thermal Factor,** C_t $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$
Table 1.5-1 of ASCE 7-16	**Importance Factor,** I_s $I_s = 0.8$ - Importance Factor *For Risk Category I.*	$I_s = 0.8$
Section 7.3 *Equation 7.3-1*	**Flat Roof Snow Load,** p_f From Section 7.3 - Equation 7.3-1, the flat roof snow load p_f can be calculated as follows: p_f - Flat Roof Snow Load $$p_f = 0.7\, C_e\, C_t\, I_s\, p_g$$ $$p_f = 194.14 \text{ psf}$$ $$\text{Where } C_e = \text{Exposure Factor} = 0.9$$ $$C_t = \text{Thermal Factor} = 1.2$$ $$I_s = \text{Importance Factor} = 0.8$$ $$p_g = \text{Ground Snow Load} = 321 \text{ psf}$$	
Section 7.10	**Rain-on-snow Surcharge Load,** p_r p_r - Rain-on-snow Surcharge Load *For $0 < p_g \leq 20$ and $\theta < \frac{W}{50}$, $p_r = 5$ psf. Otherwise, $p_r = 0$ psf. This applies only to sloped roof (balanced) snow load case.* $$p_r = 5 : 0 < p_g \leq 20, \theta < \frac{W}{50}$$ $$0 : allcases$$ $$p_r = 0 \text{ psf}$$ $$\text{Where } p_g = \text{Ground Snow Load} = 321 \text{ psf}$$ $$\theta = \text{Angle of slope of roof} = 55\ ^\circ$$ $$W = \text{Horizontal distance from eave to ridge - equal to [mathin] L [mathin] for monoslope/monopitch roof.} = 9.942 \text{ ft}$$	
Equation 7.7-1	**Snow Density,** γ γ - Snow Density $$\gamma = 0.13\, p_g + 14 \leq 30$$ $$\gamma = 30 \text{ lbf/ft}^3$$ $$\text{Where } p_g = \text{Ground Snow Load} = 321 \text{ psf}$$	
Table 7.3-2 of ASCE 7-16 *Figure 7-2c*	**Roof Slope Factor (Balanced),** C_s $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.* $\theta = 55\ ^\circ$ - Angle of slope of roof $C_s = 0.27273$ - Roof Slope Factor *For roof pitch angle equal to 55.000° and Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$ $\theta = 55\ ^\circ$ $C_s = 0.27273$
Section 7.4 *Section 7.4*	**Sloped Roof Snow Load,** p_s p_s - Sloped Roof Snow Load *Assumed to act on the horizontal projection of the surface* $$p_s = C_s\, p_f$$ $$p_s = 52.947 \text{ psf}$$ $$\text{Where } p_f = \text{Flat Roof Snow Load} = 194.14 \text{ psf}$$ $$C_s = \text{Roof Slope Factor} = 0.27273$$ p_s - Sloped Roof Snow Load with Rain Load *Assumed to act on the horizontal projection of the surface* $$p_s = C_s\, p_f + p_r$$ $$p_s = 52.947 \text{ psf}$$ $$\text{Where } p_f = \text{Flat Roof Snow Load} = 194.14 \text{ psf}$$ $$C_s = \text{Roof Slope Factor} = 0.27273$$ $$p_r = \text{Rain-on-snow Surcharge Load} = 0 \text{ psf}$$	

		
Section 7.3.4	**Minimum Roof Snow Load,** p_m $p_m = 0\ \mathrm{psf}$ - Minimum Roof Snow Load *For monoslope, gable, or hip roof with pitch angle* $\theta \geq 15^\circ$.	$p_m = 0\ \mathrm{psf}$

Project Details

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

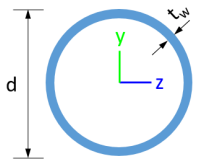
User Name: sales@mtsolar.us
Project Name: Wayman-v2cu-sq - revised mod size
Unit System: imperial

Design Input Information

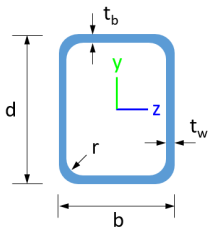
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

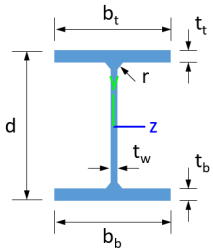
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
7	6in Pipe Sch 40	6.63	0.28					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

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)

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	27.51	27.51	13.10	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
8	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1
11	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09	300	200	1
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.09,1.08,1.09,1.08	300	200	1
15	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	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	58.34	42.30	42.30	75.35	75.35
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	102.39	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	102.39	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	24.88	6.12	40.24	43.62

14	133.20	85.85	24.82	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62

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	0.216	0.884	0.000	0.035	0.000	0.943	#13	0.735	Not Required	Pass
2	0.001	0.403	0.131	0.104	0.022	0.519	#21	0.053	Not Required	Pass
3	0.020	0.495	0.164	0.048	0.049	0.670	#21	0.045	Not Required	Pass
4	0.020	0.489	0.171	0.048	0.041	0.650	#21	0.080	Not Required	Pass
5	0.020	0.308	0.170	0.048	0.040	0.342	#21	0.074	Not Required	Pass
6	0.020	0.495	0.164	0.048	0.049	0.670	#21	0.045	Not Required	Pass
7	0.020	0.308	0.170	0.048	0.040	0.342	#21	0.074	Not Required	Pass
8	0.000	0.021	0.130	0.017	0.018	0.151	#21	Not Required	Not Required	Pass
9	0.003	0.016	0.047	0.001	0.000	0.065	#21	0.136	Not Required	Pass
10	0.020	0.489	0.171	0.048	0.041	0.650	#21	0.080	Not Required	Pass
11	0.000	0.021	0.130	0.017	0.018	0.151	#21	Not Required	Not Required	Pass
12	0.001	0.403	0.131	0.104	0.022	0.519	#21	0.053	Not Required	Pass
13	0.007	0.113	0.456	0.032	0.034	0.554	#21	0.190	Not Required	Pass
14	0.011	0.116	0.456	0.032	0.034	0.555	#21	0.190	Not Required	Pass
15	0.000	0.021	0.130	0.017	0.018	0.151	#21	Not Required	Not Required	Pass
16	0.000	0.021	0.130	0.017	0.018	0.151	#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

REFERENCES	CALCULATIONS	RESULTS

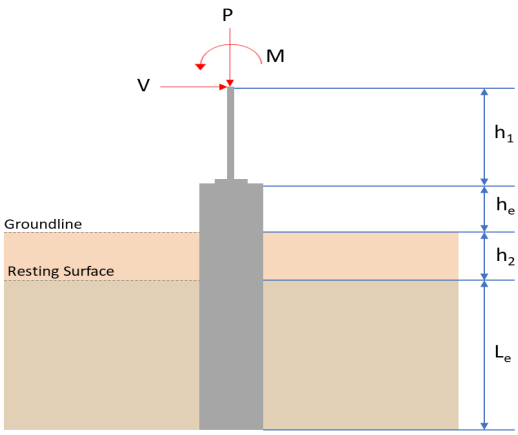
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 4.5\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - 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	Sedimentary & foliated rock	4000.000	400.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.733	12.627
V_x (kip)	-1.588	-2.646
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	21.212	37.380

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{1.57\ D}$$

$$H_o = \frac{(-1.588\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.25287\ \text{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\,H)}{1.57\,D}$$

$$M_o = \frac{(21.212\ \text{kipft}) + ((-1.588\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.3777\ \text{kipft/ft}$$

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:
$L_{e,x} = 4.2564\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(4.2564\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 4.256\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 4.5\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.256\ \text{ft})}{(4.5\ \text{ft})}$$

$$Ratio = 0.94578$$

Status: **PASS**
Ratio: **0.950**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (48\ \text{in}) \times (48\ \text{in})$$

$$A = 16\ \text{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.733\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.48331\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.48331\ \text{kip/ft}^2)}{(4000\ \text{psf})}$$

$$Ratio = 0.12083$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.25287\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.3777\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3777\ \text{kipft/ft}) \times (4.5\ \text{ft})) + \left(3 \times (-0.25287\ \text{kip/ft}) \times (4.5\ \text{ft})^2\right)}{(6 \times (3.3777\ \text{kipft/ft})) + (4 \times (-0.25287\ \text{kip/ft}) \times (4.5\ \text{ft}))}$$

$$a = 3.0688\ \text{ft}$$

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

$$p = \frac{0.75\ [(4\ M_o) + (3\ H_o\ L_e)]^2}{L_e^2\ [(3\ M_o) + (2\ H_o\ L_e)]}$$

$$p = \frac{0.75 \times [(4 \times (3.3777\ \text{kipft/ft})) + (3 \times (-0.25287\ \text{kip/ft}) \times (4.5\ \text{ft}))]^{\ 2}}{(4.5\ \text{ft})^2 \times [(3 \times (3.3777\ \text{kipft/ft})) + (2 \times (-0.25287\ \text{kip/ft}) \times (4.5\ \text{ft}))]}$$

$$p = 0.48057\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{6\ [(2\ M_o) + (H_o\ L_e)]}{L_e^2}$$

$$s = \frac{6 \times [(2 \times (3.3777\ \text{kipft/ft})) + ((-0.25287\ \text{kip/ft}) \times (4.5\ \text{ft}))]}{(4.5\ \text{ft})^2}$$

$$s = 1.6644\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400\ \text{psf/ft}) \times \frac{(3.0688\ \text{ft})}{2}$$

$$p_a = 0.61375\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.48057\ \text{kip/ft}^2\right)}{\left(0.61375\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.783$$

Status: **PASS**
Ratio: **0.780**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\ L_e$$

$$p_s = (400\ \text{psf/ft}) \times (4.5\ \text{ft})$$

$$p_s = 1.8\ \text{kip/ft}^2$$

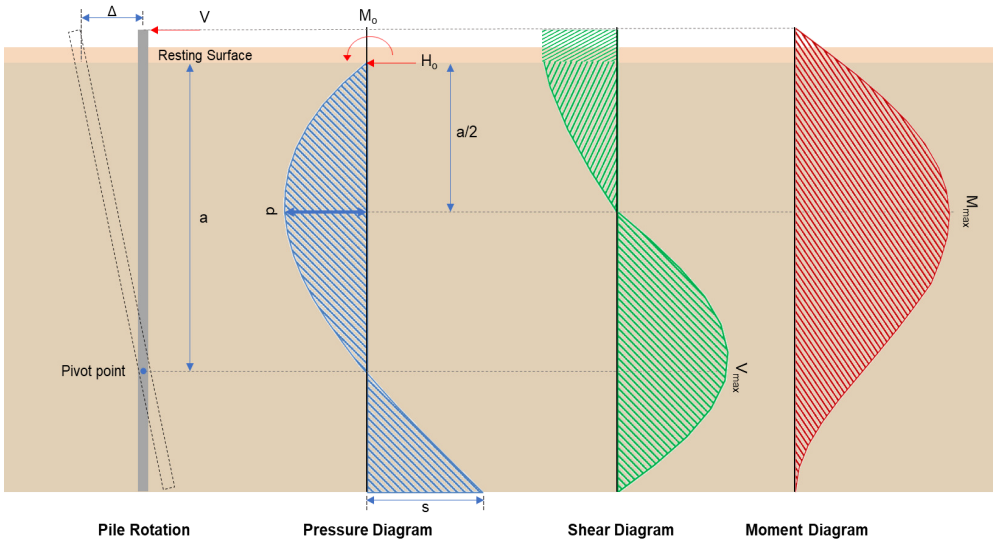
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.6644\ \text{kip/ft}^2\right)}{\left(1.8\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.92469$$

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



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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{1.57\,D}$$

$$H_o = \frac{(-2.646\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.42134\text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\,H)}{1.57\,D}$$

$$M_o = \frac{(37.38\text{ kipft}) + ((-2.646\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 5.9522\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.9522\text{ kipft/ft})}{(-0.42134\text{ kip/ft})}$$

$$E = 14.127\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.9522\text{ kipft/ft}) \times (4.5\text{ ft})) + \left(3 \times (-0.42134\text{ kip/ft}) \times (4.5\text{ ft})^2\right)}{(6 \times (5.9522\text{ kipft/ft})) + (4 \times (-0.42134\text{ kip/ft}) \times (4.5\text{ ft}))}$$

$$a = 3.0657\text{ ft}$$

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

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

$$V_{max} = ((-0.42134\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.127\text{ ft})}{(4.5\text{ ft})} + 3\right) \times \left(\frac{(3.0657\text{ ft})}{(4.5\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (14.127\text{ ft})}{(4.5\text{ ft})} + 2\right) \times \left(\frac{(3.0657\text{ ft})}{(4.5\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 10.484\text{ kip}$$

$$V_{max} = 10.484 \text{ kip}$$

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

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

$$M_{max} = ((-0.42134 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(14.127 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0657 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.127 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0657 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.127 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0657 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.895 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min \left[\frac{\frac{(12.627 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.177 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\, [(-84.177 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

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

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

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(2304\ \text{in}^2\right) - \left(4.2951\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(4.2951\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 2675.2\ \text{kip}$$

$Ratio$ - Capacity

$$Ratio = \frac{P}{\phi P_N}$$

$$Ratio = \frac{(12.627\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0047201$$

Status: **PASS**
Ratio: **0.000**

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (48\ \text{in})$$

$$d = 38.4\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,max} = 296.21\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 12.627\ \text{kip} \rightarrow 12627\ \text{lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(12627\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 120.17\ \text{kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

$$V_{c,b} = \left[2\ \lambda_s\ \sqrt{f'_{ck}} + \left(0.05\ f'_{ck}\right)\right]\ b_w\ d$$

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + (0.05 \times (2500\ \text{psi}))\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,b} = 348.89\ \text{kip}$$

V_c - Governing shear strength of concrete

$$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$

$$V_c = Min\left[(296.21\ \text{kip}), (120.17\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 120.17\ \text{kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{s,a} = 737.28\ \text{kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$

$$V_{s,b} = 50.894\ \text{kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$

$$V_s = 50.894\ \text{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.17\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.19\ \text{kip}$$

Considering x-direction:

$V_{max} = 10.484\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.484\ \text{kip})}{(111.19\ \text{kip})}$$

$$Ratio = 0.094285$$

Status: **PASS**
Ratio: **0.090**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b\,D^2}{6}$$

$$S_m = \frac{(48\text{ in}) \times (48\text{ in})^2}{6}$$

$$S_m = 18432\text{ in}^3$$

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

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$

$$\phi M_{n,1} = 249.600kipft$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\text{ ksi}) \times \left(18432\text{ in}^3\right)$$

$$\phi M_{n,2} = 2121.6\text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(249.6\text{ kipft}), (2121.6\text{ kipft})]$$

$$\phi M_n = 249.6\text{ kipft}$$

Considering x-direction:

$M_{max} = 22.895\text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(22.895\text{ kipft})}{(249.6\text{ kipft})}$$

$$Ratio = 0.091726$$

Status: **PASS**
Ratio: **0.090**