Your Project Calculations

Project Name: Dana CB-6

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Dana%20CB-6&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=K8I66M1VHa8jTWvF04tHqdrq1Ukueib2sNWpHpY5GsaNFtFlam5CRy1y8F01hAP2

Array Specification

Product:	Cyber
Unique ID:	1P-0-6TOP-SD-24-L-3Hx2W-0BIA
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	68.00in
Number of Rows:	3
Number of Columns:	2
Total Number of Modules:	6
Desired Tilt Angle:	47
Front Edge Clearance:	6
Total Array Height at Tilt:	13.36 ft
Total Frame Length:	11.50 ft
Frame Weight:	352 lbs
Array Dimensions N/S:	10.13 ft
Array Dimensions E/W:	11.50 ft
Rail Length:	121.50 in
Rail Spacing:	2.88 ft
Rail Check:	PASS (36% utilized)

Support Specifications

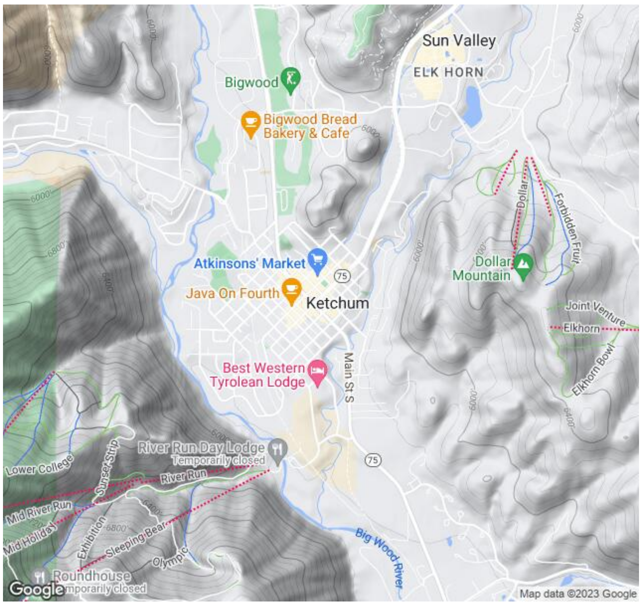
Pole Size:	4in Pipe Sch 80
Pole Length above Grade:	9.70 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 3.75 ft
Foundation Volume:	2.222 y^3
Foundation Result:	PASSED
Mount Twist:	0.000003 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	HGX2+X2X, Ketchum, ID, USA
Wind Speed:	97 mph
Snow Load:	170.89 psf
Design Uplift Pressure:	0.009334 ksf
Design Downforce Pressure:	-0.009334 ksf
Design Snow Pressure:	0.043221 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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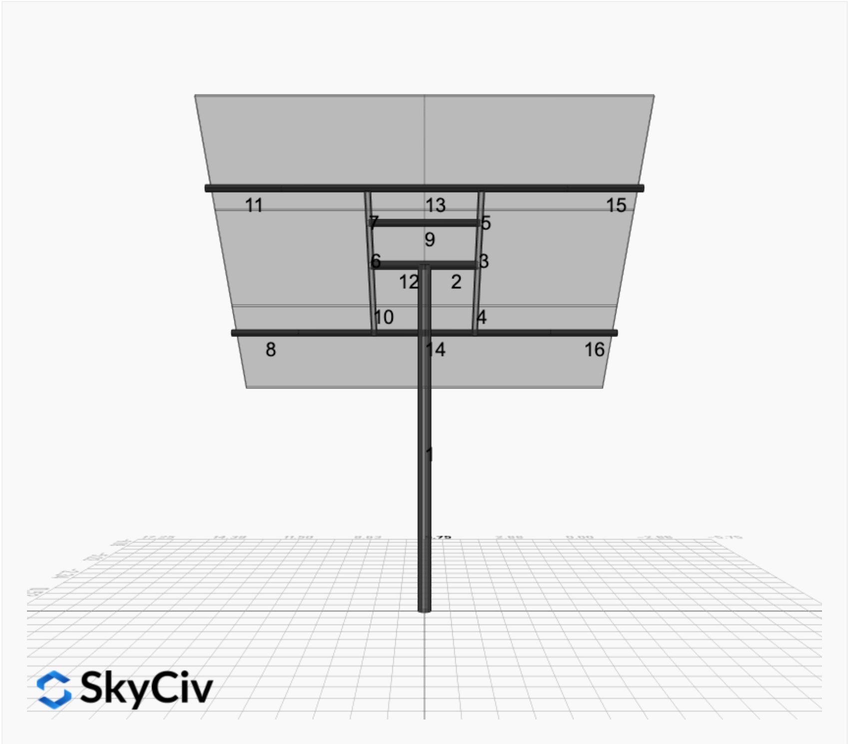
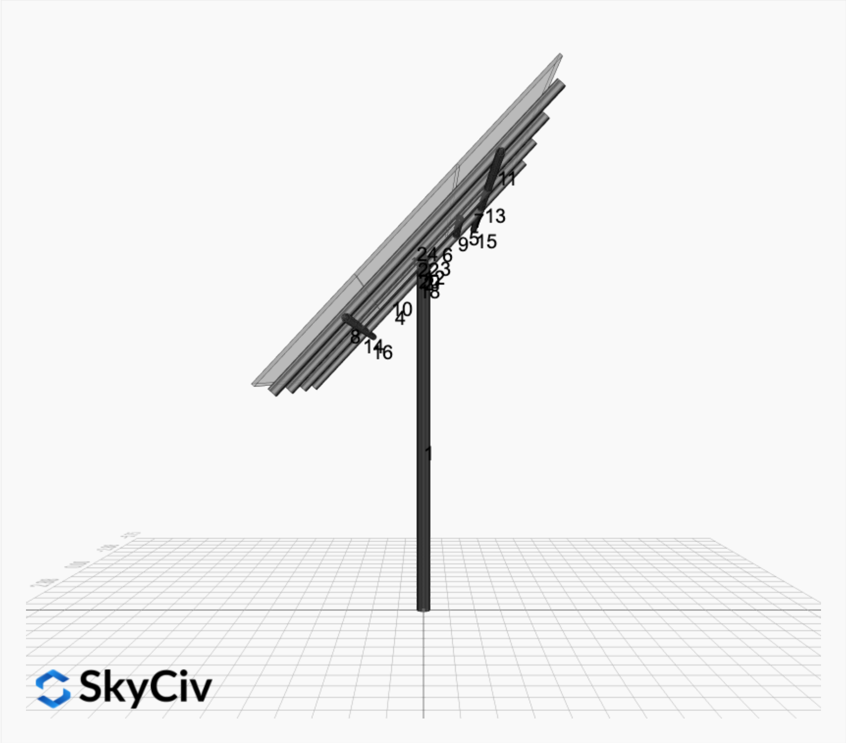
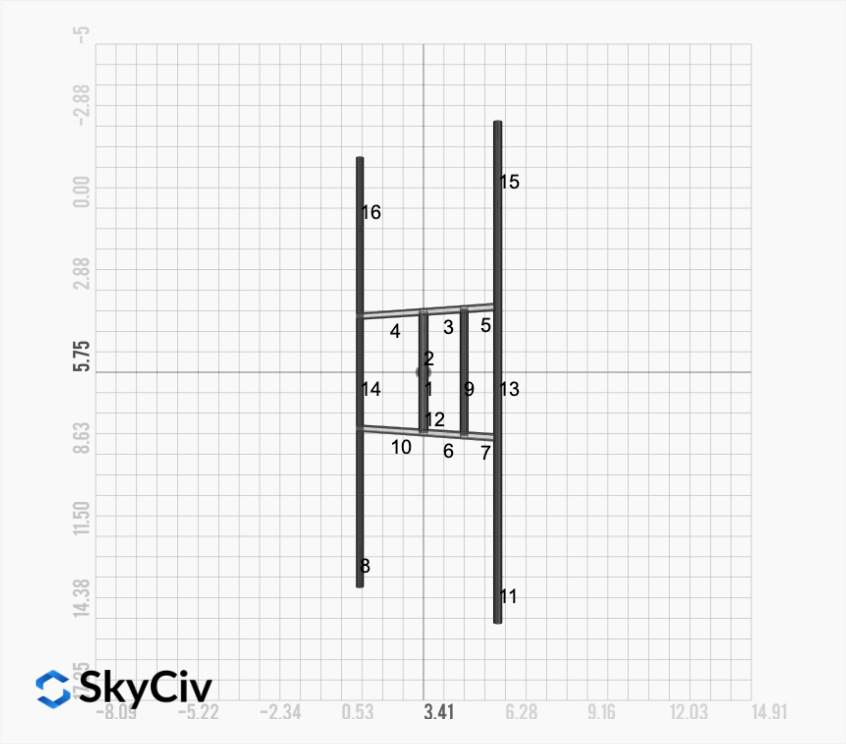
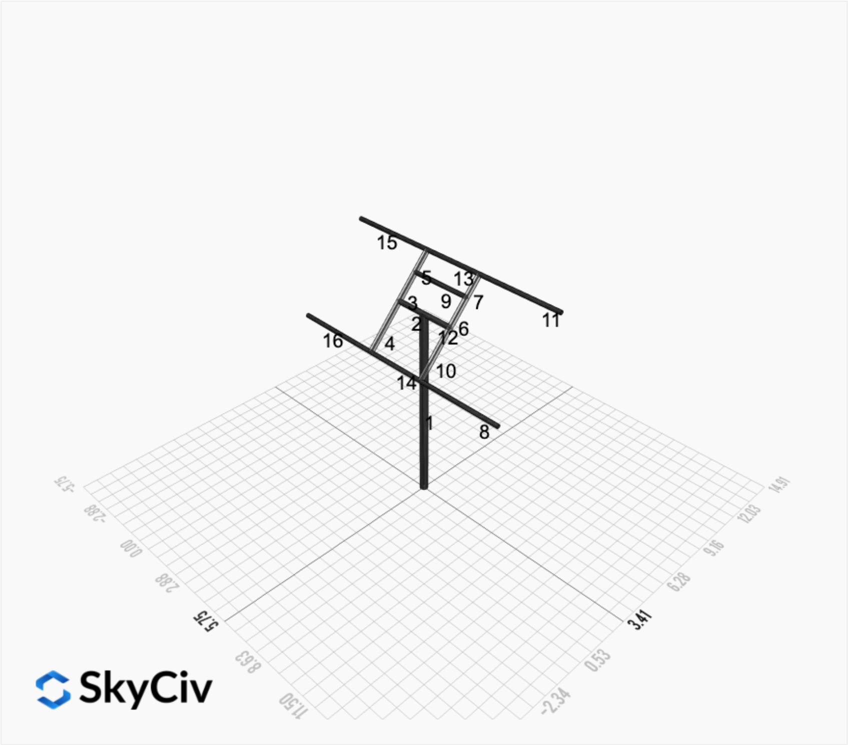
Design Notes:

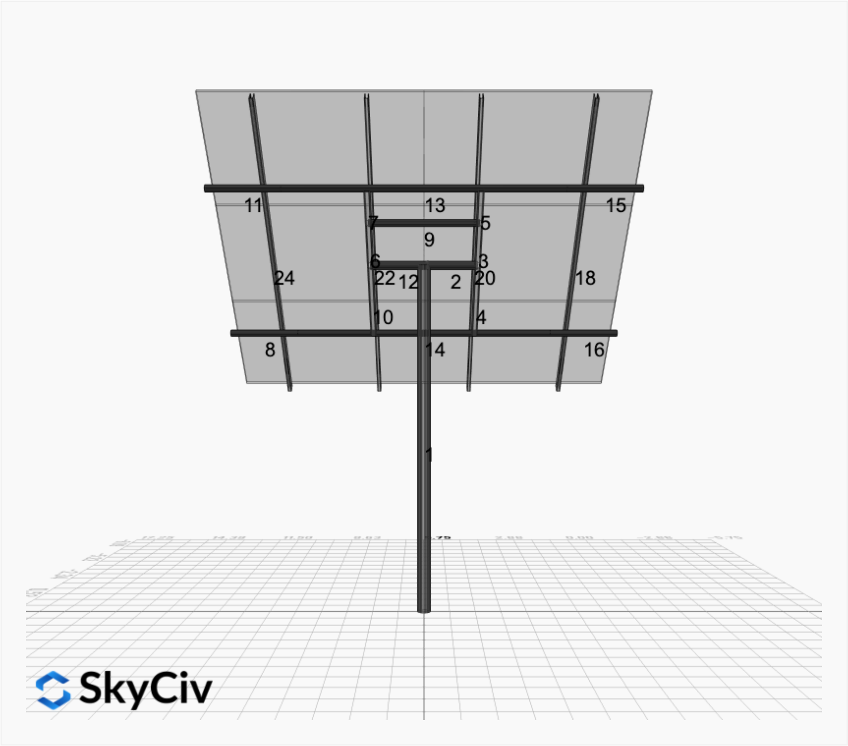
- AISC Deflection checks are set to L/1 due to structure design intent




SkyCiv

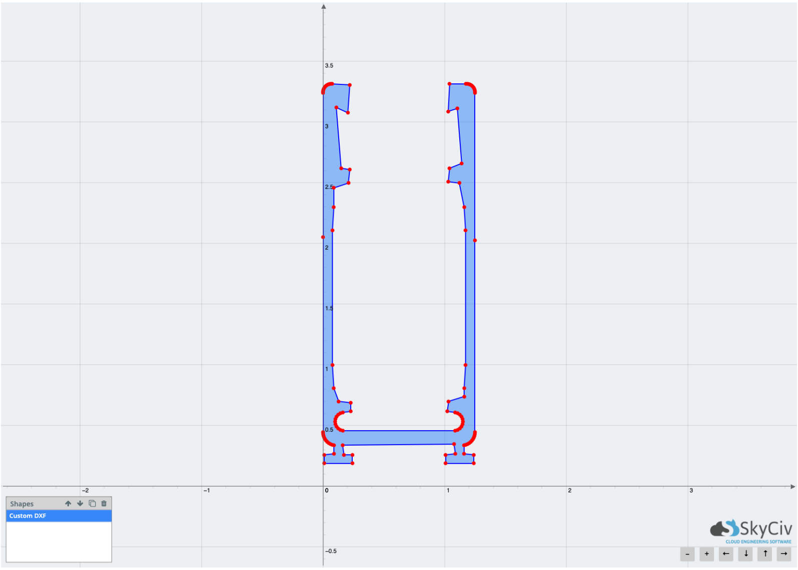

-5
-2.88
0.00
2.88
5.75
8.63
11.50
14.38
-8.09
-5.22
-2.34
0.53
3.41
6.28
9.16
12.03
14.91
SkyCiv


15
5
13
3
9
7
2
6
11
12
16
4
10
14
1
8
-5.75
-2.88
0.00
2.88
5.75
8.63
11.50
14.91
12.03
9.16
6.28
3.41
0.53
-2.34
SkyCiv

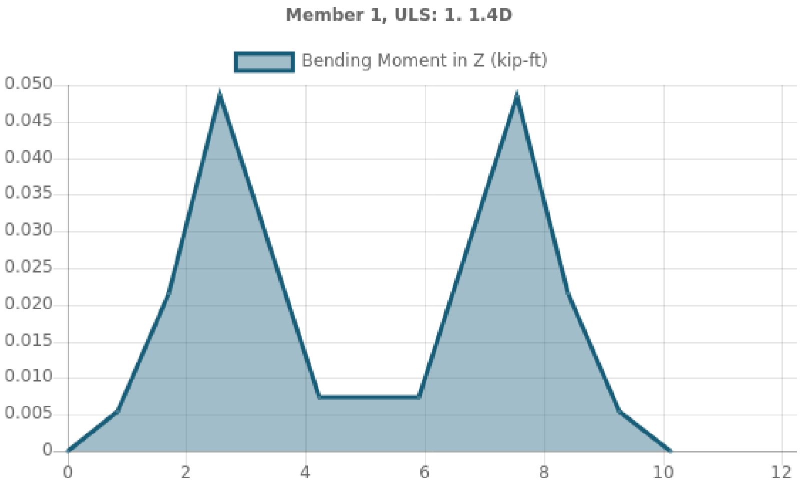

11
13
15
7
5
9
6
3
24
22
12
2
20
18
10
4
8
14
16
1
SkyCiv

Rail Design Check

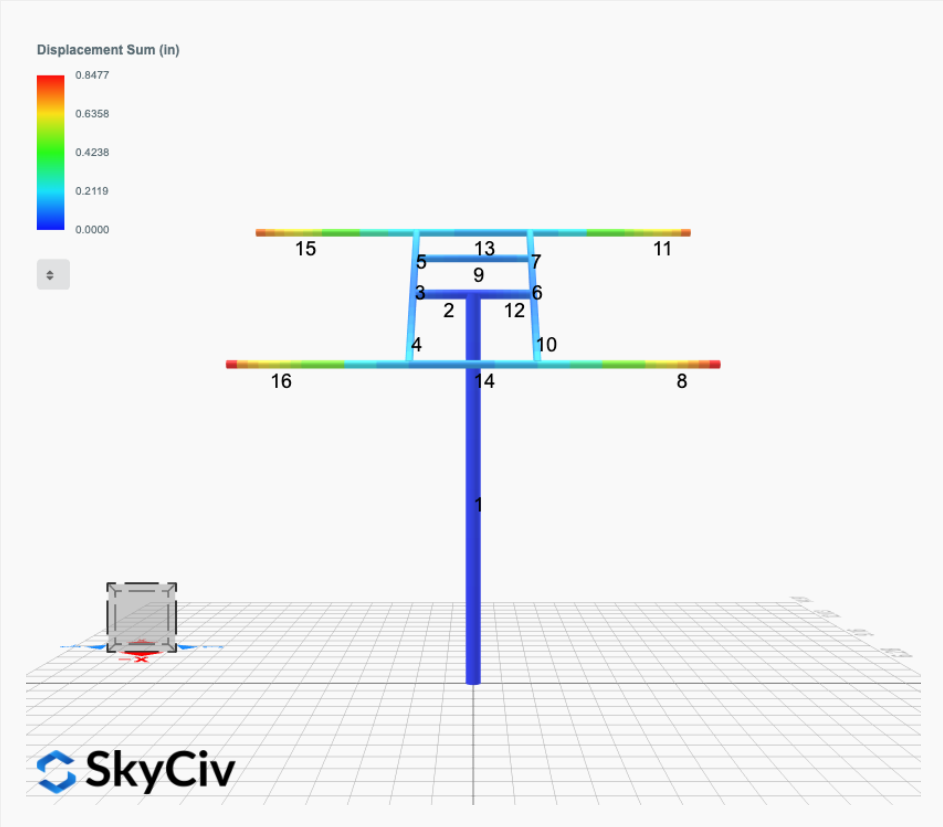
Rail Length: 10.125 ft
Additional Restraints Required: None
Tributary Width: 2.875 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0847 kip/ft
Snow (Y): -0.0909 kip/ft
Wind uplift Case A: 0.0268 kip/ft
Wind downforce Case A: 0.0268 kip/ft
Dead (Panel load) (X): 0.0099 kip/ft
Dead (Panel load) (Y): -0.0106 kip/ft

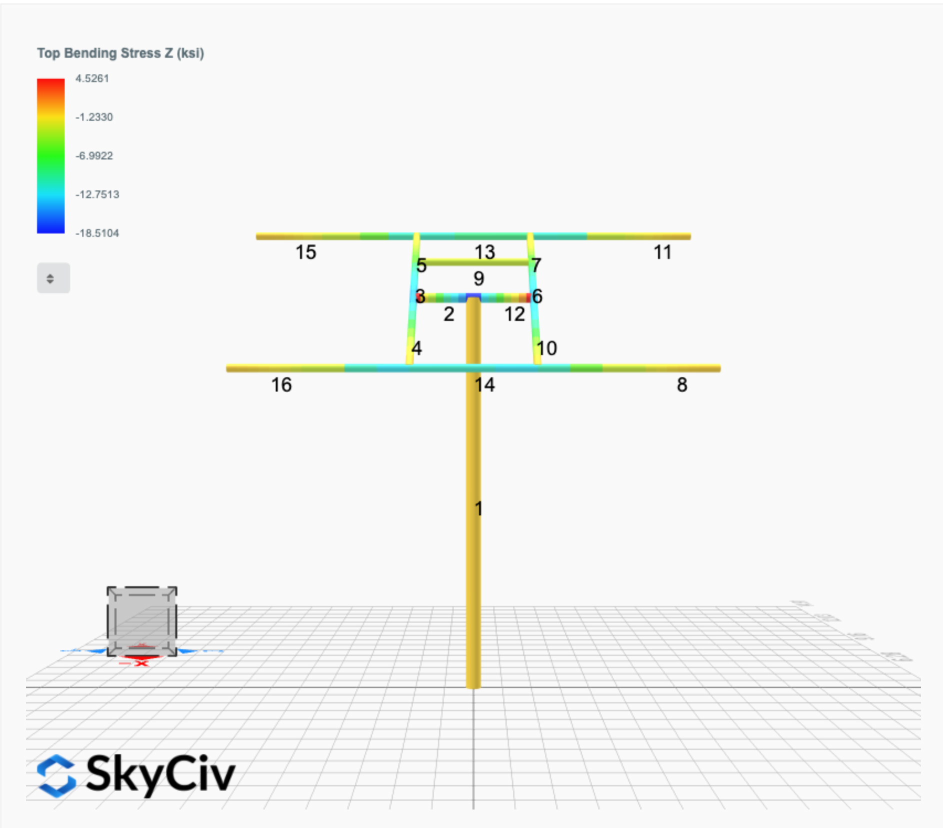


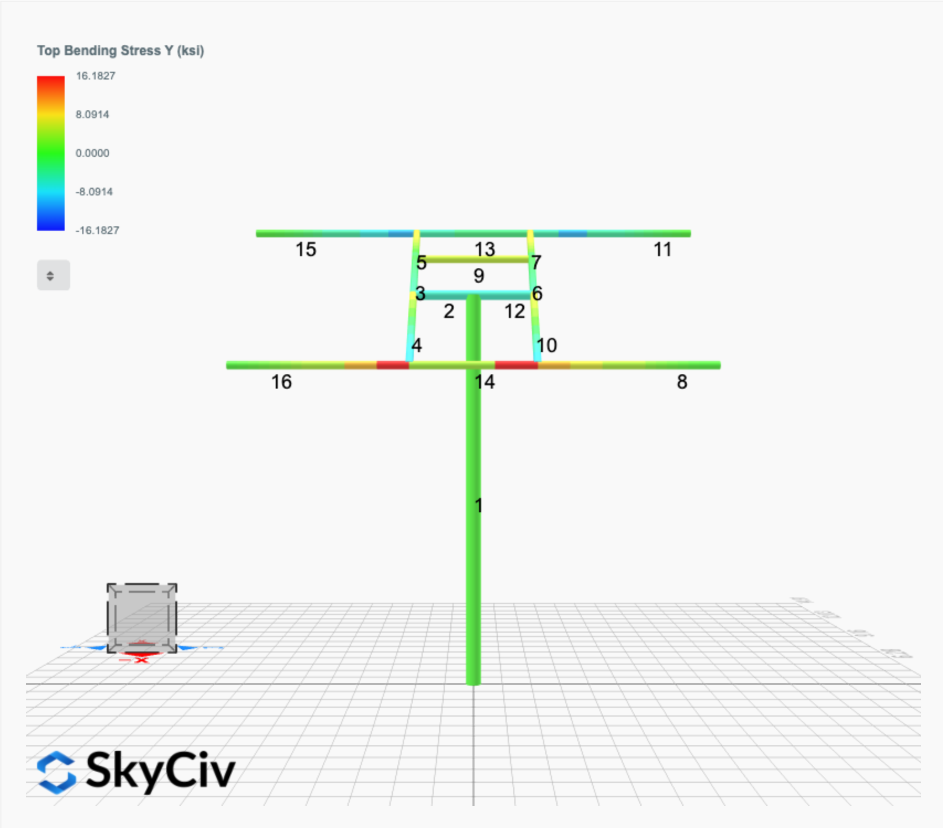
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	12.59066611	0.365	PASS
Material Yield	34.5	12.59066611	0.365	PASS
Material Strength	37	12.59066611	0.340	PASS



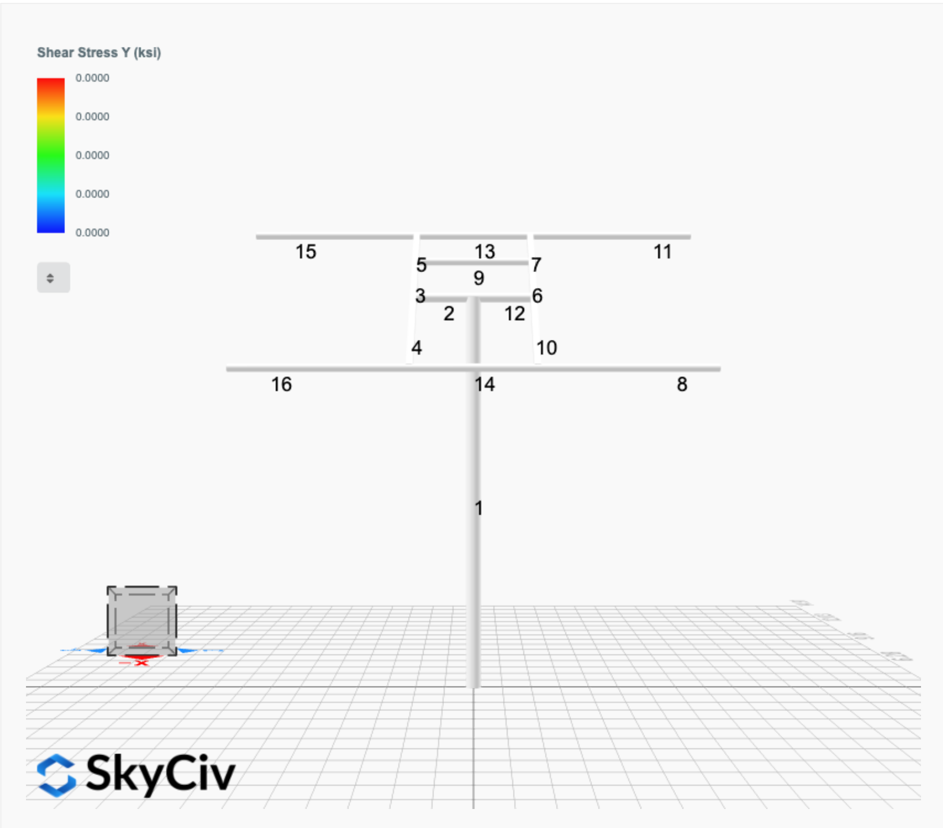
FEM Results (Envelope Worst Case for each member)



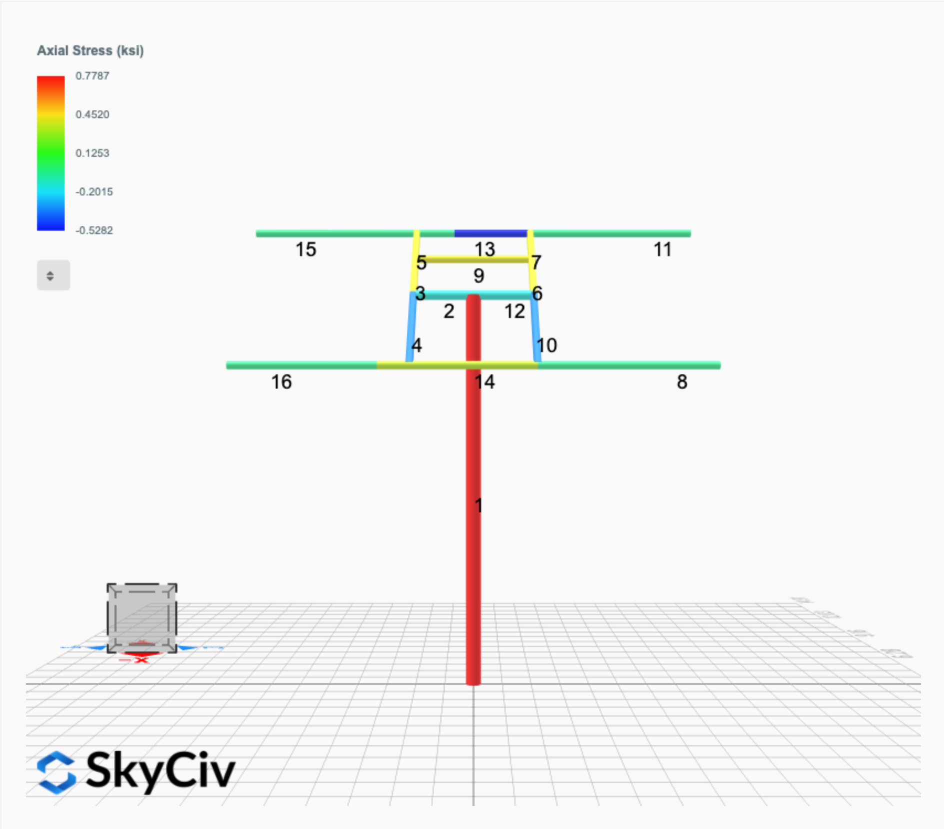




Top Bending Stress Y (ksi)
16.1827
8.0914
0.0000
-8.0914
-16.1827
15
13
11
5
9
7
3
6
2
12
4
10
16
14
8
1
SkyCiv



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


Axial Stress (ksi)
0.7787
0.4520
0.1253
-0.2015
-0.5282
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	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 2. D + L	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 3. D + (S or Lr or R)	0.0000	4.3695	0.0000	0.0000	-0.0000	0.0388
ULS: 3. D + (S or Lr or R)	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.5115	0.0000	0.0000	-0.0000	0.0329
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 5b. D + 0.7E	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.5115	0.0000	0.0000	-0.0000	0.0329
ULS: 8. 0.6D + 0.7E	-0.0000	0.5624	0.0000	0.0000	-0.0000	0.0092
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.4769	1.3821	0.0000	0.0000	-0.0000	4.6989
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4769	0.4927	0.0000	0.0000	-0.0000	-4.5569
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3577	3.8450	0.0000	0.0000	-0.0000	3.5456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.5115	0.0000	0.0000	-0.0000	0.0329
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3577	3.1780	0.0000	0.0000	-0.0000	-3.3963
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.5115	0.0000	0.0000	-0.0000	0.0329
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3577	1.2709	0.0000	0.0000	-0.0000	3.5280
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3577	0.6038	0.0000	0.0000	-0.0000	-3.4139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	0.9374	0.0000	0.0000	-0.0000	0.0154
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.4769	1.0071	-0.0000	0.0000	-0.0000	4.6928
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.5624	0.0000	0.0000	-0.0000	0.0092
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4769	0.1177	0.0000	0.0000	-0.0000	-4.5631
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.5624	0.0000	0.0000	-0.0000	0.0092

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	6.9869
Shear X	-0.7948
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	8.2348

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	4.3695
Shear X	-0.4769
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	4.6989

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : Dana CB-6
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 **HGX2+X2X, Ketchum, ID, USA** 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 **7318 ft** above mean sea level.

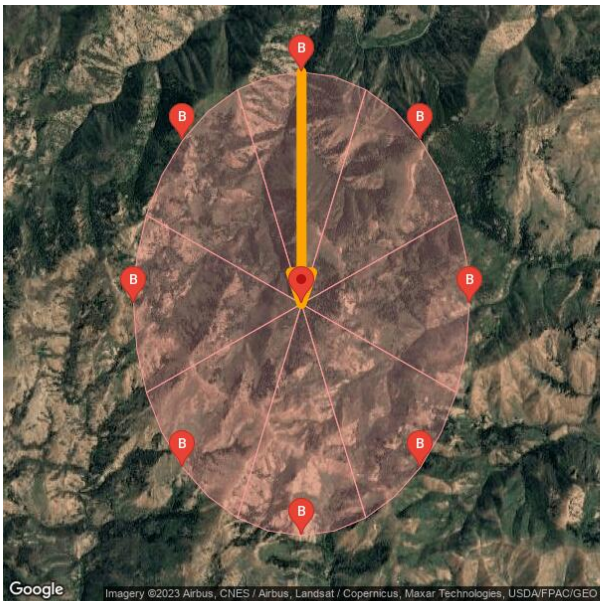


Figure 1. Site location.

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

Parameter	Value
Ground to Top of Wall/Sign, h	13.359 ft
Wall/Sign Horizontal Dimension, B	11.500 ft
Wall/Sign Vertical Dimension, s	7.314 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 **97 mph** 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.	$V = 97$ mph
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 (greater than 1.0) if the location satisfies all of the following conditions: If $H/L_h > 0.2$ $$H/L_h = 1894.7779537811684/13023.340 = 0.145$$ Since $H/L_h < 0.2$, $K_{zt} = 1.0$.	$K_{zt} = 1.0$
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 7317.66 ft above mean sea level. To account for air density, K_e is calculated in accordance with Table 26.9-1 using the formula: $$K_e = e^{-0.0000362 z_g}$$ $$K_e = e^{-0.0000362(7317.66)} = 0.767$$	$K_e = 0.767$

Velocity Pressure Exposure Coefficient, K_z and Velocity Pressure, q_z

Section 26.10
Table 26.10-1

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

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

$$q_z = 0.00256 K_z (1)(0.85)(0.767)(97)^2$$

where: K_z is calculated for each height using Table 27.3-1 rounded to nearest hundredth. The table below shows the **comparison of 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 @ 13.359 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	0.570	1.000	0.850	0.767	97.000	8.954
S	B	0.570	1.000	0.850	0.767	97.000	8.954
E	B	0.570	1.000	0.850	0.767	97.000	8.954
W	B	0.570	1.000	0.850	0.767	97.000	8.954
NE	B	0.570	1.000	0.850	0.767	97.000	8.954
SE	B	0.570	1.000	0.850	0.767	97.000	8.954
NW	B	0.570	1.000	0.850	0.767	97.000	8.954
SW	B	0.570	1.000	0.850	0.767	97.000	8.954

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

1.000

Level	Height, ft	K_z	q_z, psf
Ground to Top of Wall/Sign	13.359	0.57	8.95

Net Force Coefficient, C_f

Figure 29.3-1 of ASCE 7-16

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:

$$B/s = 11.50/7.31 = 1.572$$

$$s/h = 7.31/13.36 = 0.547$$

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:

$$C_{f,A} = R_{factor,open} C_{f,A} = (1.000)(1.698) = 1.698$$

For Case B:

$$C_{f,B} = R_{factor,open} C_{f,B} = (1.000)(1.698) = 1.698$$

Equation 29.3-1 of ASCE 7-16

Design wind Force, F

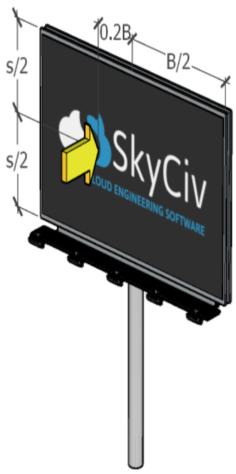
The design wind force, F, can be calculated using Equation 29.3-1 of ASCE 7-16.

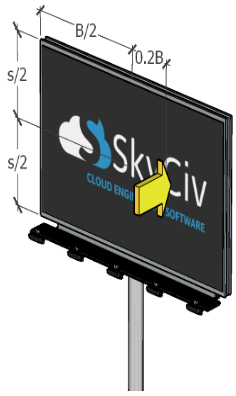
$$F = q_h G C_f A_s = (8.95)(0.85)C_f(84.11) = 640.184C_f$$

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

Case	Location	C_f	Design Force, F lb
Case A	e = 0 ft	1.698	1086.799
Case B	e = 2.30 ft	1.698	1086.799

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 : Dana CB-6
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **HGX2+X2X, Ketchum, ID, 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 **7318 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	6.820 ft
Building Width, B	11.500 ft
Mean Roof Height, h	9.702 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	47.000°

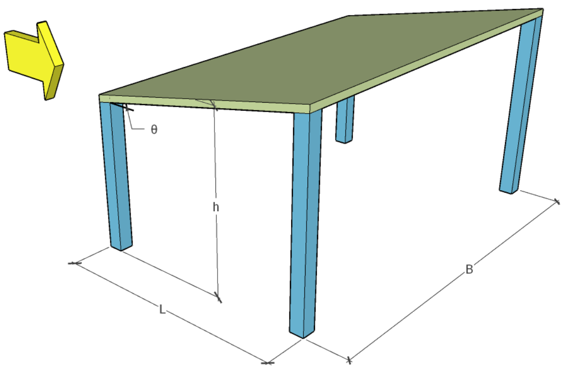


Figure 2. Building parameters.

Ground Snow Load, p_g

Section 7.2 of ASCE 7

The ground snow load, p_g, for the site location is **170.89 psf** at elevation 7317.66 ft above mean sea level based on Section 7.2 of ASCE 7.

$p_g = 170.89$ psf

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: $$p_f = 0.7 C_e C_t I_s p_g$$ $$p_f = 0.7(0.90)(1.20)(0.80)(170.89) = 103.35 psf$$	$p_f = 103.35$ 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: $$\gamma = 0.13 p_g + 14 \leq 30 = 0.13(170.89) + 14 \leq 30$$ $$\gamma = 30.00 pcf$$	$\gamma = 30.00 pcf$
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 0.418 based on Figure 7.2c where $\theta = 47.00°$.	$C_s = 0.418$
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: $$p_s = C_s p_f$$ $$p_s = (0.418)(103.35) = 43.22 psf$$	$p_s = 43.22$ psf

Project Details

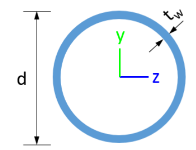
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

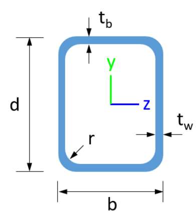
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
28	2.5in Pipe Sch 80	2.88	0.28					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
30	HSS3x2x3/16	3.00	2.00	0.17	0.17	0.17		

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
28	2.5in Pipe Sch 80	2.25	3.85	1.92	1.92	0.00	1.87	1.87
30	HSS3x2x3/16	1.54	2.05	0.93	1.77	1.95	1.12	1.48

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	5	20.38	20.38	9.70	-	300	200	1
2	28	0.98	0.98	1.50	-	30	20	1

						0	0	
3	30	0.92	0.92	1.42	1.31,1.30,1.31,1.29,1.30,1.31,1.30,1.30,1.31,1.30,1.30,1.31,1.27,1.31,1.29,1.29,1.29,1.29,1.30,1.31,1.47,1.31,1.30,1.31,1.28,1.31	300	200	1
4	30	1.63	1.63	2.50	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.92,1.68,1.67,1.68,1.66,1.68	300	200	1
5	30	0.70	0.70	1.08	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.81,1.67,1.67,1.67,1.66,1.67	300	200	1
6	30	0.92	0.92	1.42	1.31,1.30,1.31,1.29,1.30,1.31,1.30,1.30,1.31,1.30,1.30,1.31,1.27,1.31,1.29,1.29,1.29,1.29,1.30,1.31,1.47,1.31,1.30,1.31,1.28,1.31	300	200	1
7	30	0.70	0.70	1.08	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.80,1.67,1.67,1.67,1.66,1.67	300	200	1
8	2	4.20	4.20	2.00	-	300	200	1
9	2	1.95	1.95	3.00	-	300	200	1
10	30	1.63	1.63	2.50	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.92,1.68,1.67,1.68,1.66,1.68	300	200	1
11	2	4.20	4.20	2.00	-	300	200	1
12	28	0.98	0.98	1.50	-	300	200	1
13	2	4.88	3.00	7.50	-	300	200	1
14	2	4.88	3.00	7.50	-	300	200	1
15	2	4.20	4.20	2.00	-	300	200	1
16	2	4.20	4.20	2.00	-	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	198.33	36.32	21.95	21.95	59.50	59.50
2	101.41	100.23	7.02	7.02	30.42	30.42
3	69.30	68.29	5.55	4.20	23.28	13.89
4	69.30	66.19	5.55	4.20	23.28	13.89
5	69.30	68.70	5.55	4.20	23.28	13.89
6	69.30	68.29	5.55	4.20	23.28	13.89
7	69.30	68.70	5.55	4.20	23.28	13.89
8	66.48	48.46	3.82	3.82	19.94	19.94
9	66.48	62.10	3.82	3.82	19.94	19.94
10	69.30	66.19	5.55	4.20	23.28	13.89
11	66.48	48.46	3.82	3.82	19.94	19.94
12	101.41	100.23	7.02	7.02	30.42	30.42
13	66.48	43.42	3.82	3.82	19.94	19.94
14	66.48	43.42	3.82	3.82	19.94	19.94
15	66.48	48.46	3.82	3.82	19.94	19.94
16	66.48	48.46	3.82	3.82	19.94	19.94

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	0.192	0.375	0.000	0.013	0.000	0.424	#13	0.828	Not Required	Pass
2	0.006	0.592	0.195	0.112	0.013	0.790	#21	0.042	Not Required	Pass
3	0.017	0.562	0.118	0.053	0.021	0.689	#21	0.071	Not Required	Pass
4	0.017	0.553	0.299	0.053	0.063	0.780	#21	0.084	Not Required	Pass
5	0.017	0.243	0.326	0.052	0.103	0.335	#21	0.054	Not Required	Pass
6	0.017	0.562	0.118	0.053	0.021	0.689	#21	0.071	Not Required	Pass
7	0.017	0.243	0.326	0.052	0.103	0.335	#21	0.054	Not Required	Pass
8	0.000	0.111	0.106	0.021	0.020	0.216	#21	Not Required	Not Required	Pass
9	0.018	0.112	0.159	0.001	0.000	0.279	#21	0.153	Not Required	Pass
10	0.017	0.553	0.299	0.053	0.063	0.780	#21	0.084	Not Required	Pass
11	0.000	0.111	0.106	0.021	0.020	0.216	#21	Not Required	Not Required	Pass
12	0.006	0.592	0.195	0.112	0.013	0.790	#21	0.042	Not Required	Pass
13	0.022	0.500	0.477	0.045	0.043	0.977	#21	0.254	Not Required	Warn
14	0.020	0.500	0.477	0.045	0.043	0.977	#21	0.254	Not Required	Warn
15	0.000	0.111	0.106	0.021	0.020	0.216	#21	Not Required	Not Required	Pass
16	0.000	0.111	0.106	0.021	0.020	0.216	#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/comresssion)
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

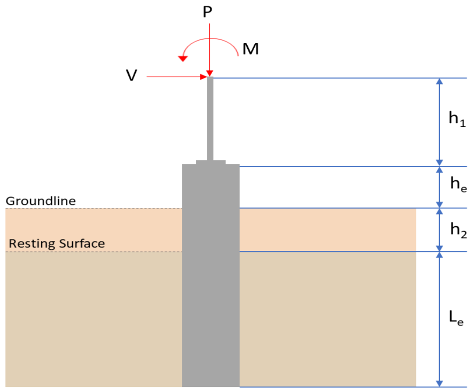
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 3.75\ \mathrm{ft}$ - 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)	4.370	6.987
V_x (kip)	-0.477	-0.795
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	4.699	8.235

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,

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

$$H_o = \frac{(-0.477\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.075955\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(4.699\ \mathrm{kipft}) + ((-0.477\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.74825\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 3.5249\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(3.5249\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 3.525\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (3.75\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 3.75\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(3.525\ \mathrm{ft})}{(3.75\ \mathrm{ft})}$$

$$Ratio = 0.94$$

Status: PASS
Ratio: 0.940

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.37\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.27313\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.27313\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.13656$$

Status: PASS
Ratio: 0.140

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(3.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 0.9375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.075955\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.74825\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.74825\ \mathrm{kipft/ft}) \times (3.75\ \mathrm{ft})) + \left(3 \times (-0.075955\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft})^2\right)}{(6 \times (0.74825\ \mathrm{kipft/ft})) + (4 \times (-0.075955\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))}$$

$$a = 2.5633\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.74825\ \mathrm{kipft/ft})) + (3 \times (-0.075955\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]^2}{(3.75\ \mathrm{ft})^2 \times [(3 \times (0.74825\ \mathrm{kipft/ft})) + (2 \times (-0.075955\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]}$$

$$p = 0.14561\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.74825\ \mathrm{kipft/ft})) + ((-0.075955\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]}{(3.75\ \mathrm{ft})^2}$$

$$s = 0.51698\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(2.5633\ \mathrm{ft})}{2}$$

$$p_a = 0.19224\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.14561\ \mathrm{kip/ft^2}\right)}{\left(0.19224\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.7574$$

Status: **PASS**
Ratio: **0.760**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (3.75\ \mathrm{ft})$$

$$p_s = 0.5625\ \mathrm{kip/ft^2}$$

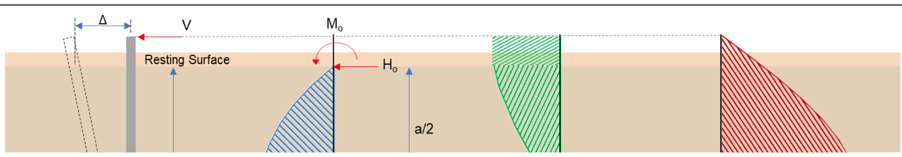
$Ratio$ - Lateral soil capacity

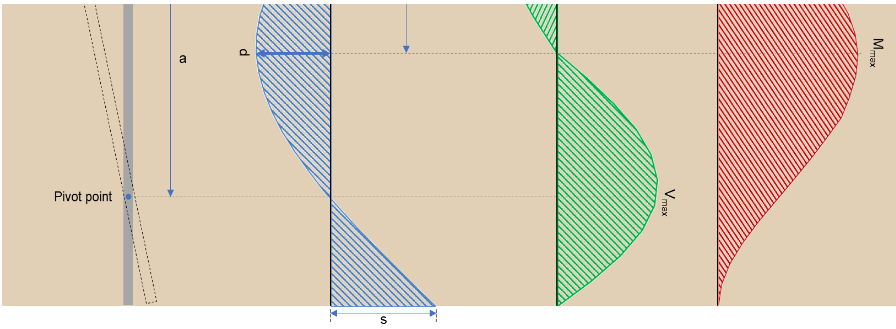
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.51698\ \mathrm{kip/ft^2}\right)}{\left(0.5625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.91907$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.795\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.12659\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(8.235\text{ kipft}) + ((-0.795\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 1.3113\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.3113\text{ kipft/ft})}{(-0.12659\text{ kip/ft})}$$

$$E = 10.358\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.3113\text{ kipft/ft}) \times (3.75\text{ ft})) + \left(3 \times (-0.12659\text{ kip/ft}) \times (3.75\text{ ft})^2\right)}{(6 \times (1.3113\text{ kipft/ft})) + (4 \times (-0.12659\text{ kip/ft}) \times (3.75\text{ ft}))}$$

$$a = 2.5608\text{ ft}$$

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

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

$$V_{max} = ((-0.12659\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.358\text{ ft})}{(3.75\text{ ft})} + 3\right) \times \left(\frac{(2.5608\text{ ft})}{(3.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.358\text{ ft})}{(3.75\text{ ft})} + 2\right) \times \left(\frac{(2.5608\text{ ft})}{(3.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 2.811\text{ kip}$$

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

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

$$M_{max} = ((-0.12659\text{ kip/ft}) \times (48\text{ in}) \times (3.75\text{ ft})) \times \left[\left(\frac{(10.358\text{ ft})}{(3.75\text{ ft})} + \frac{(2.5608\text{ ft})}{2 \times (3.75\text{ ft})}\right) - \left[\left(\frac{4 \times (10.358\text{ ft})}{(3.75\text{ ft})} + 3\right) \times \left(\frac{(2.5608\text{ ft})}{2 \times (3.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.358\text{ ft})}{(3.75\text{ ft})} + 2\right) \times \left(\frac{(2.5608\text{ ft})}{2 \times (3.75\text{ ft})}\right)^4\right]\right]$$

	$$M_{max} = 5.0972 \text{ kipft}$$		

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 $\alpha = 0.8$ - 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(6.987 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -102.03 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-102.03 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

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 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**	

22.4.2.2

Axial Compression Strength (ACI 318-19, LRFD)

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (3\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$

$$\phi P_N = 3183.4\ \mathrm{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.987\ \mathrm{kip})}{(3183.4\ \mathrm{kip})}$$

$$Ratio = 0.0021948$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,max} = 324.49\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $P = 6.987\ \mathrm{kip} \rightarrow 6987\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + \frac{(6987\ \mathrm{lbf})}{6 \times \left(2304\ \mathrm{in}^2\right)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 130.73\ \mathrm{kip}$$

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

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + (0.05 \times (3000\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,b} = 406.27\ \mathrm{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49\ \mathrm{kip}), (130.73\ \mathrm{kip}), (406.27\ \mathrm{kip})\right]$$

$$V_c = 130.73\ \mathrm{kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 807.65 \text{ kip}$$

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

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

$$V_s = MIN\left[(807.65 \text{ kip}), (50.894 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((130.73 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 118.05 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.811 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(2.811 \text{ kip})}{(118.05 \text{ kip})}$$

$$Ratio = 0.023811$$

Status: **PASS**
Ratio: **0.020**

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:
$\phi M_{n,1}$

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

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

$$\phi M_{n,1} = 273.423 kipft$$

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

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

$$\phi M_{n,2} = \phi\ 0.85\ f_{ck}\ S_m$$

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

$$\phi M_{n,2} = 2545.9\ \mathrm{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(273.42\ \mathrm{kipft}), (2545.9\ \mathrm{kipft})]$$

$$\phi M_n = 273.42\ \mathrm{kipft}$$

Considering x-direction:

$M_{max} = 5.0972\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(5.0972\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$

$$Ratio = 0.018642$$

Status: **PASS**
Ratio: **0.020**