Project Details

Project Name: SunUp Ranch - 5x2 - V1Jb

Location: 9J3F+65, East Gull Lake, MN 56401, USA

Unique ID: 1P-0-6TOP-XD-45-L-5Hx2W-STRUTS-0AH2

Dealer: ______________________________

Date: Mon Dec 02 2024

Number of Modules: 10

Number of Poles: 1

Date Sold:

____

Array Dimensions N/S	18.81 ft
Array Dimensions E/W	15.12 ft
Winter Tilt Angle	20
Front Edge Clearance	8 ft

MT Solar Bill of Materials (1P-0-6TOP-XD-45-L-5Hx2W-STRUTS-0AH2)

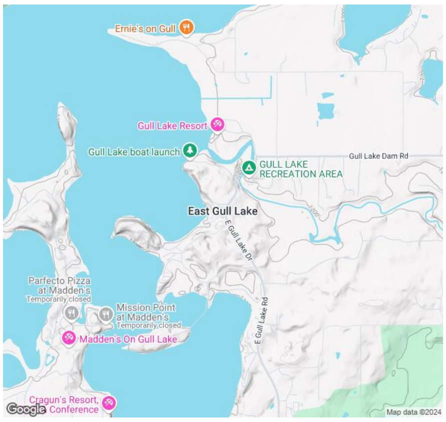
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-XD	H-Frame Assembly-XD	1
MTS-XD-Wing-45	45IN XD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	2

Rail Bill of Materials

Part	Qty
Rails (223in)	4
Rail Attachment	16
Module Mid Clamp	16
Module End Clamp	8

Part	Qty
Ground Lug	2

Site Details:



Site Address: 9J3F+65, East Gull Lake, MN 56401, USA

Array Specification

Duty Classification:	XD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	5
Number of Columns:	2
Total Number of Modules:	10
Winter Tilt Angle:	20
Front Edge Clearance:	8
Total Array Height at Tilt:	14.43 ft
Total Frame Length:	15.00 ft
Frame Weight:	1128 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	15.12 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	11.22 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: 5.50 ft
Foundation Volume:	3.259 y^3

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	9J3F+65, East Gull Lake, MN 56401, USA
Wind Speed:	102 mph
Snow Load:	60 psf

Design Disclaimer

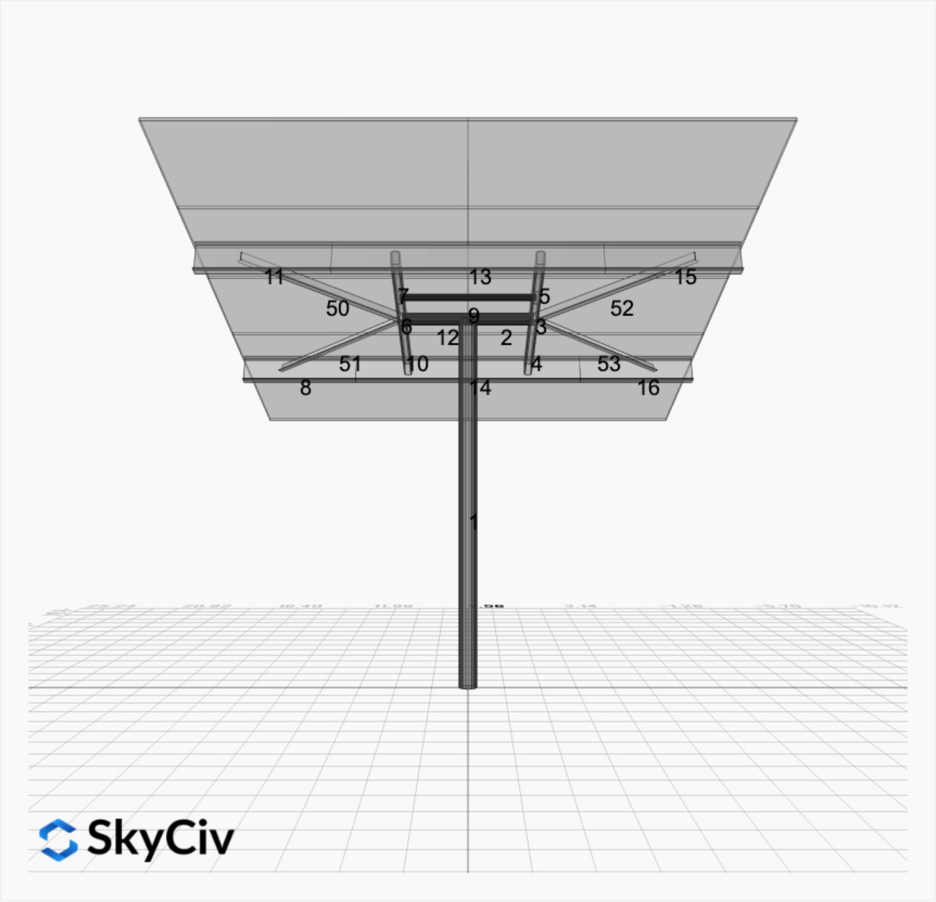
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

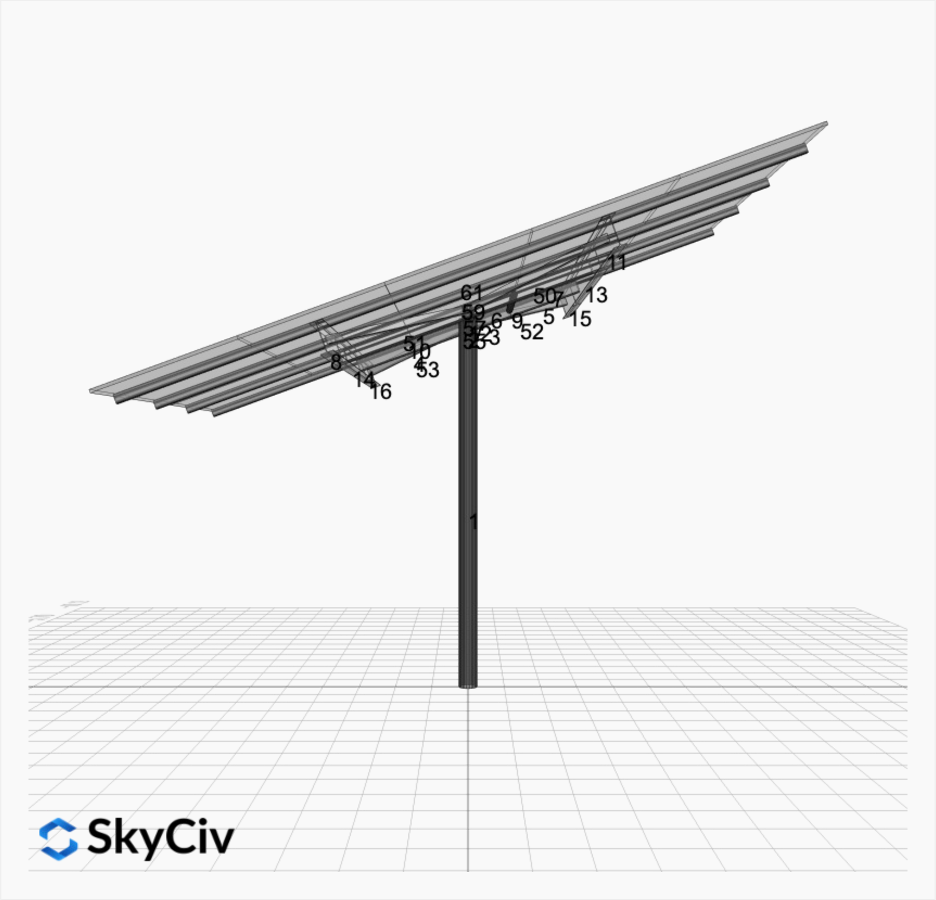
AutoDesigner Input

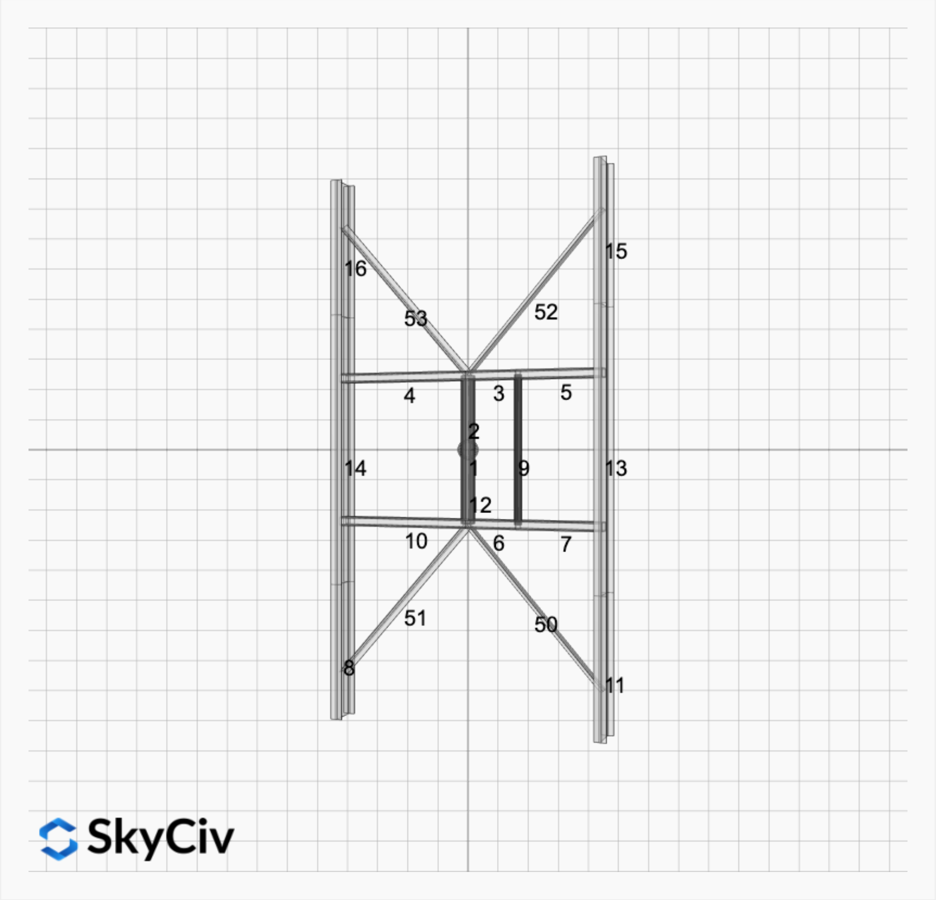

```
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":true,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"SunUp Ranch - 5x2 - V1Jb","site_address":"9J3F+65, East Gull Lake, MN 56401, USA","module_width":44.65,"module_length":89.72,"number_rows":"5","number_columns":2,"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":20,"ground_clearance":8,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":true}
```

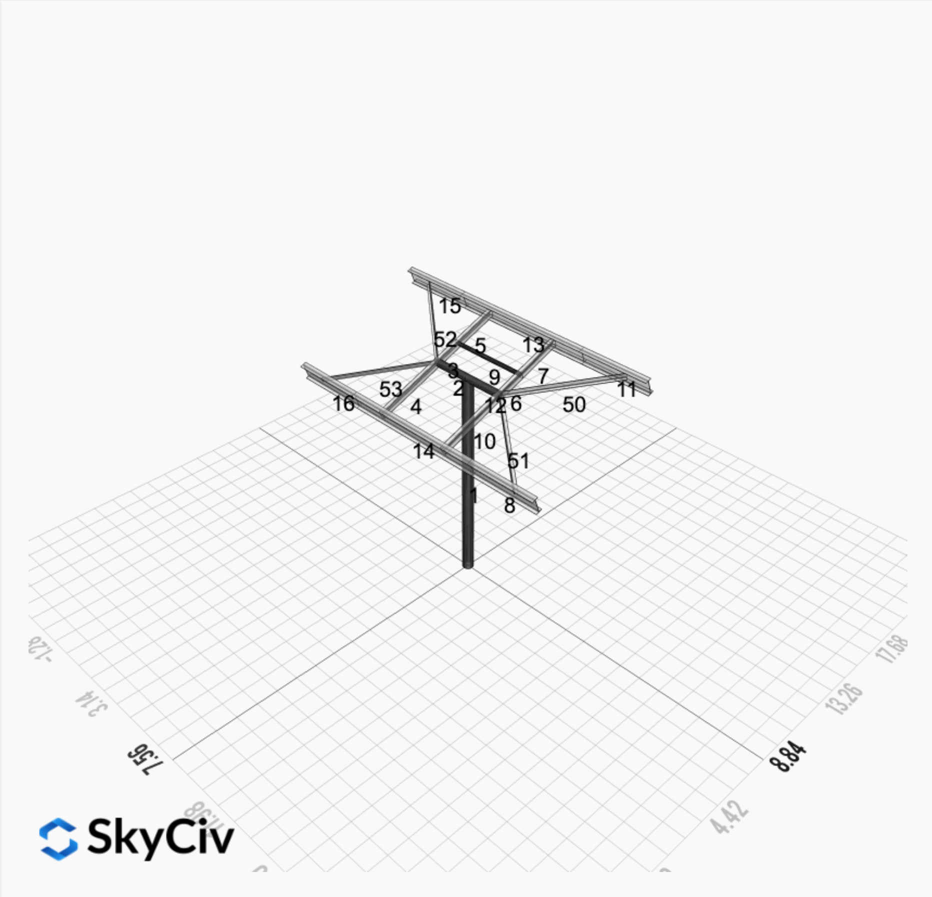

Design Notes:

- 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)




61
59
50
7
13
11
6
9
5
15
52
51
10
4
53
8
14
16
1
SkyCiv

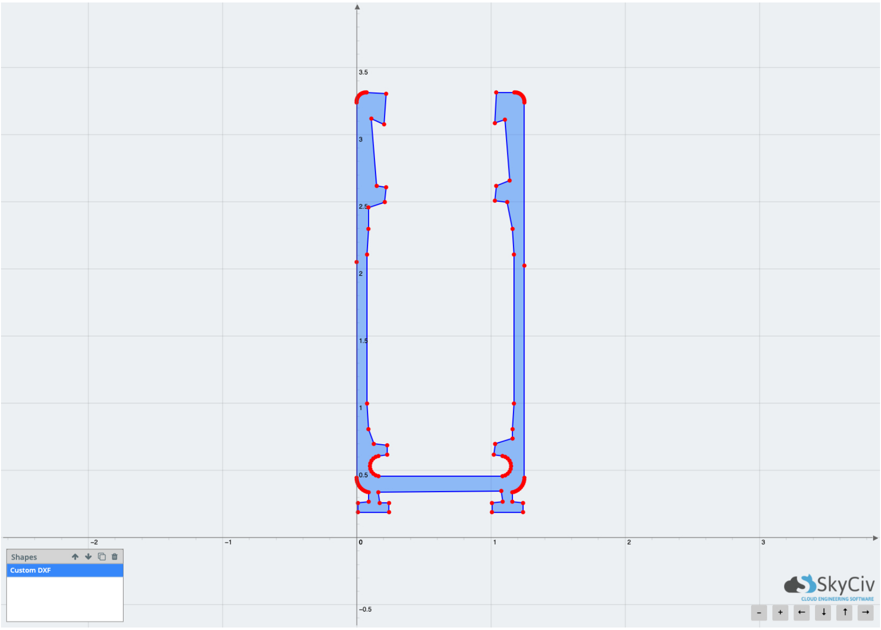

16
15
53
52
4
3
5
2
14
1
9
13
12
10
6
7
51
50
8
11
SkyCiv


15
52
5
13
3
9
7
11
53
2
16
4
12
6
50
10
14
51
1
8
-1.28
3.14
7.56
8.84
4.42
13.26
17.68
SkyCiv

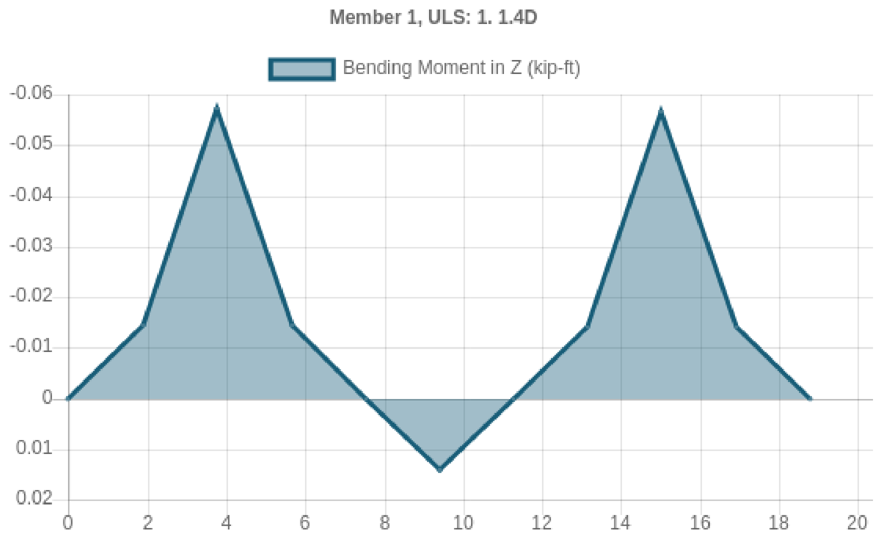

11
13
15
7
5
50
52
61
59
9
57
55
6
12
2
3
51
10
4
53
8
14
16
1
SkyCiv

Rail Design Check

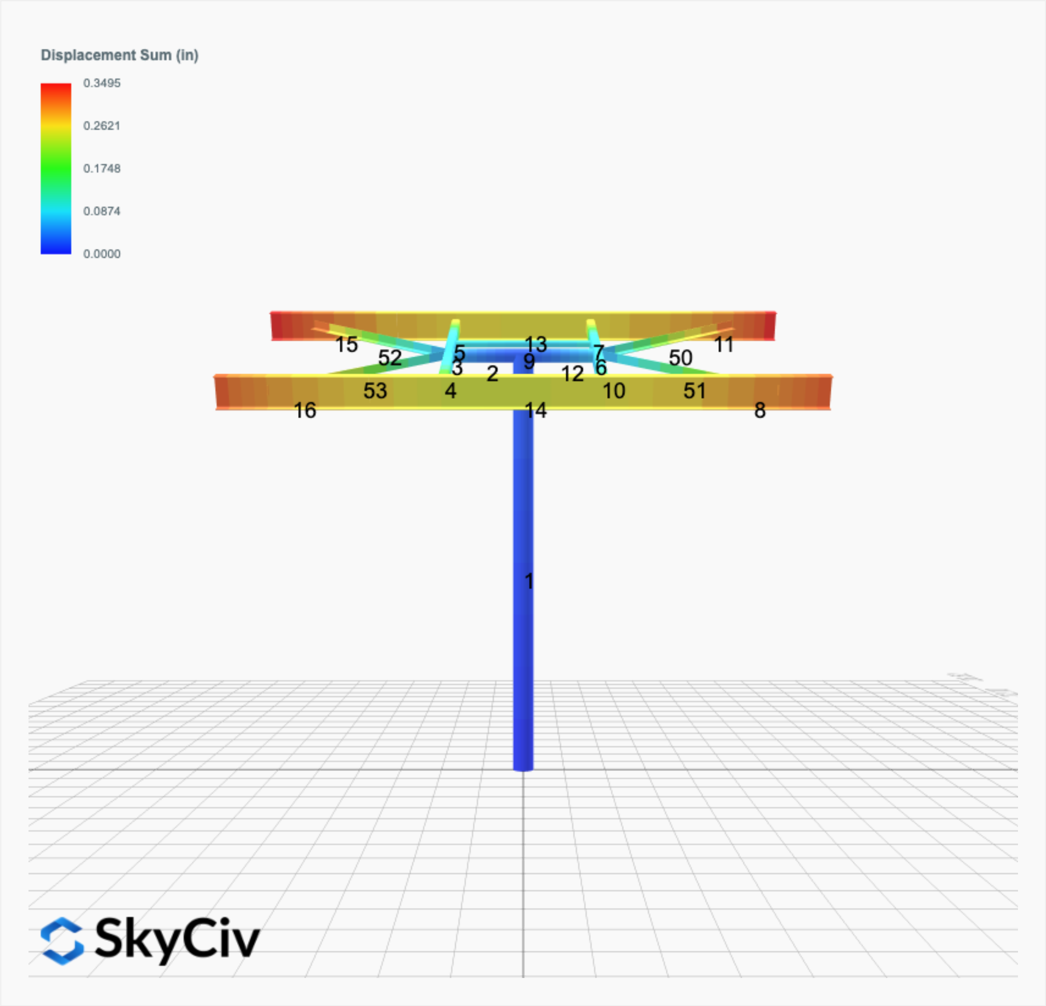
Rail Length: 18.8125 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.78 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.1172 kip/ft
Snow (Y): -0.0426 kip/ft
Wind uplift Case A: 0.0600 kip/ft
Wind uplift Case A: 0.0600 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0893 kip/ft

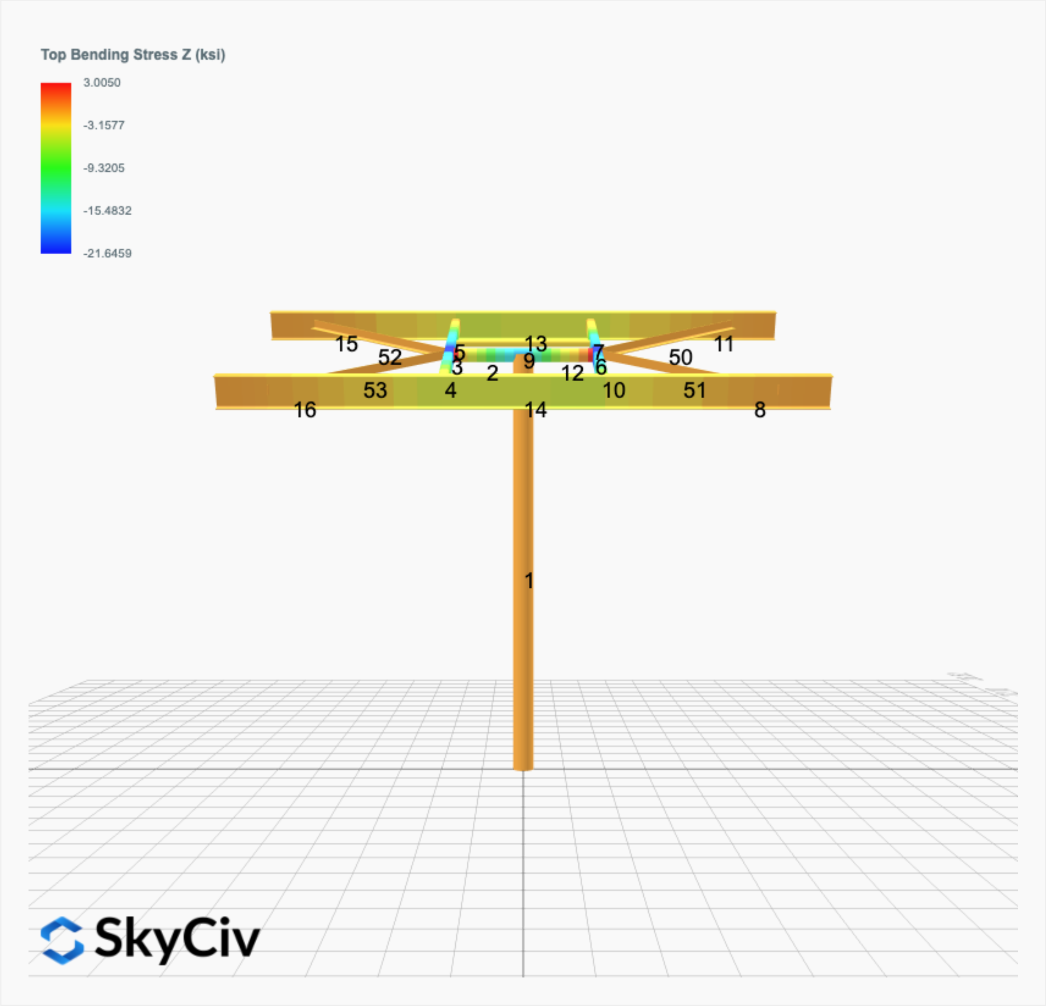


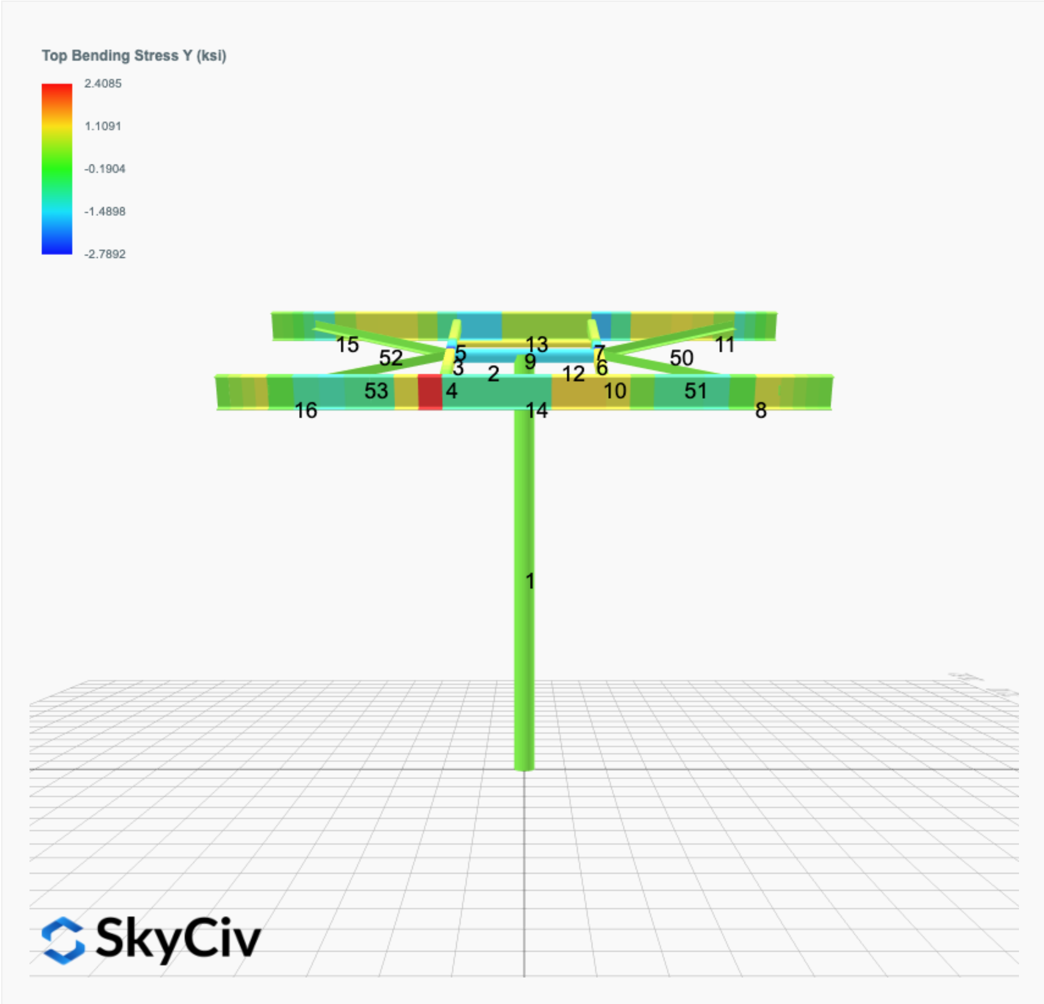
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	17.42569891	0.505	PASS
Material Yield	34.5	17.42569891	0.505	PASS
Material Strength	37	17.42569891	0.471	PASS

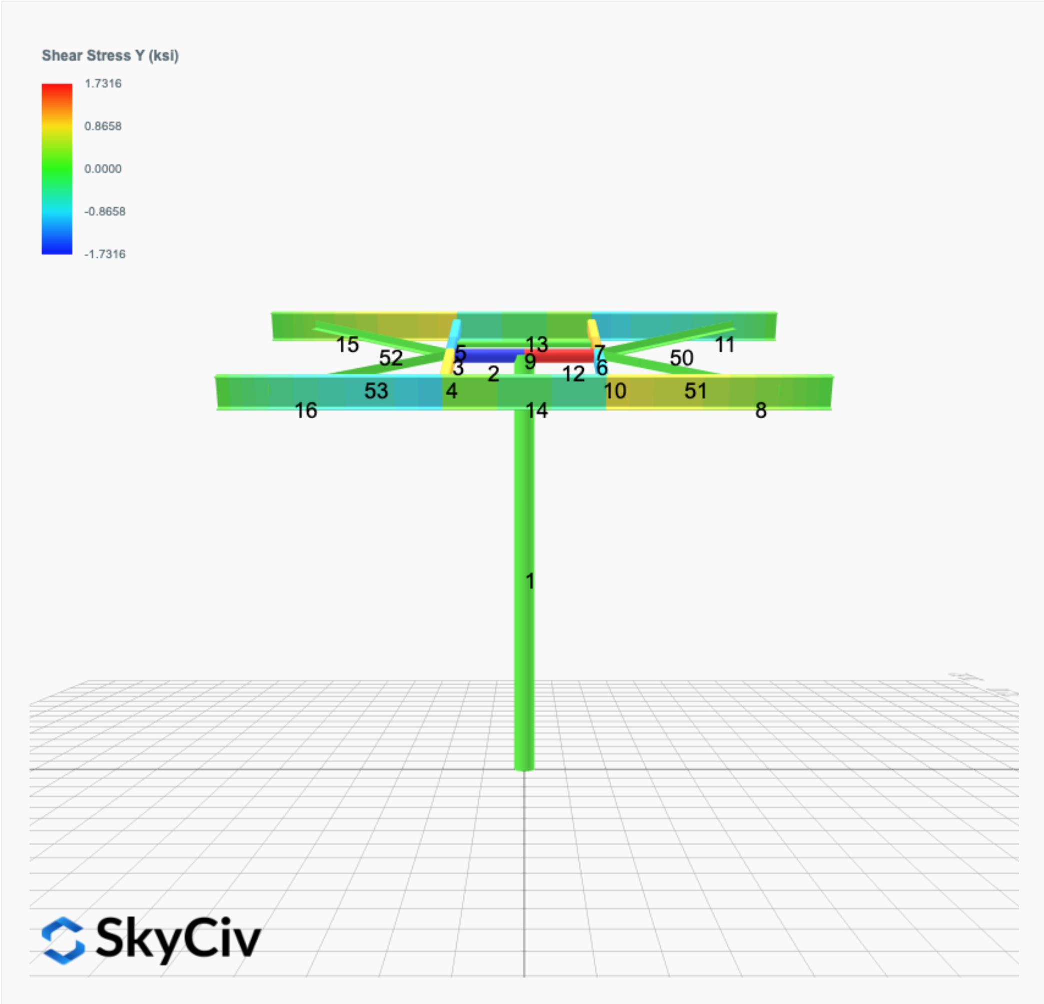


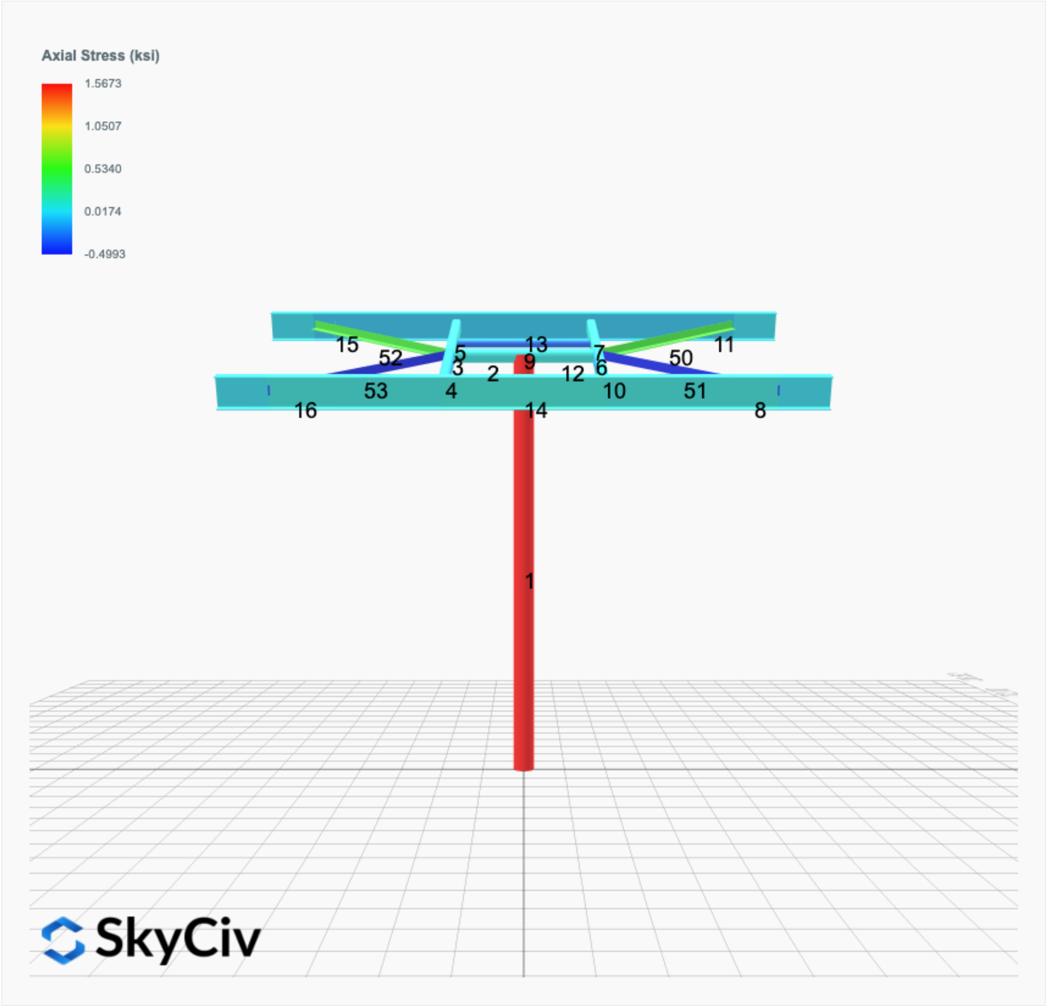
FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
3.0050
-3.1577
-9.3205
-15.4832
-21.6459
15
52
5
13
7
50
11
3
9
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Top Bending Stress Y (ksi)
2.4085
1.1091
-0.1904
-1.4898
-2.7892
15
52
5
3
13
9
2
12
7
6
50
11
53
4
10
51
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
1.7316
0.8658
0.0000
-0.8658
-1.7316
15
52
5
13
9
7
6
50
11
3
2
12
53
4
10
51
16
14
8
1
SkyCiv


Axial Stress (ksi)
1.5673
1.0507
0.5340
0.0174
-0.4993
15
52
5
3
13
9
7
6
11
50
2
12
53
4
10
51
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	2.1736	0.0000	-0.0000	-0.0000	0.0319
ULS: 2. D + L	0.0000	2.1736	0.0000	-0.0000	-0.0000	0.0319
ULS: 3. D + (S or Lr or R)	0.0000	10.9213	0.0000	-0.0000	-0.0000	0.0946
ULS: 3. D + (S or Lr or R)	0.0000	2.1736	0.0000	-0.0000	-0.0000	0.0319
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	8.7344	0.0000	-0.0000	-0.0000	0.0789
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1736	0.0000	-0.0000	-0.0000	0.0319
ULS: 5b. D + 0.7E	0.0000	2.1736	0.0000	-0.0000	-0.0000	0.0319
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	8.7344	0.0000	-0.0000	-0.0000	0.0789
ULS: 8. 0.6D + 0.7E	0.0000	1.3042	0.0000	-0.0000	-0.0000	0.0191
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0192	4.9739	0.0000	-0.0000	-0.0000	12.4717
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0192	4.9739	0.0000	-0.0000	-0.0000	12.4717
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8648	-0.2025	0.0000	-0.0000	-0.0000	-8.7456
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7516	0.1087	0.0000	-0.0000	-0.0000	-14.8209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	10.8346	0.0000	-0.0000	-0.0000	9.4088
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	10.8346	0.0000	-0.0000	-0.0000	9.4088
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	6.9523	0.0000	-0.0000	-0.0000	-6.5042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	7.1857	0.0000	-0.0000	-0.0000	-11.0607
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	4.2738	0.0000	-0.0000	-0.0000	9.3618
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	4.2738	0.0000	-0.0000	-0.0000	9.3618
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	0.3916	0.0000	-0.0000	-0.0000	-6.5512
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	0.6249	0.0000	-0.0000	-0.0000	-11.1077
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0192	4.1045	0.0000	-0.0000	-0.0000	12.4590
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0192	4.1045	0.0000	-0.0000	-0.0000	12.4590
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8648	-1.0719	0.0000	-0.0000	-0.0000	-8.7583
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7516	-0.7607	0.0000	-0.0000	-0.0000	-14.8336

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	18.9382
Shear X	-1.6987
Shear Z	0.0000
Moment X	-0.0004
Moment Y (Twist)	0.0012
Moment Z	26.3239

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	10.9213
Shear X	-1.0192
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	14.8336

Project Details

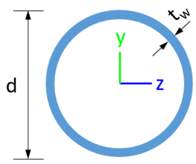
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

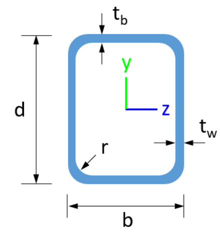
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

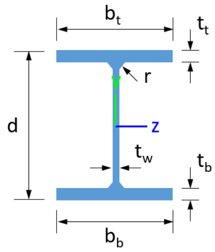
Section Dimensions



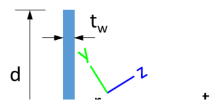
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
7	6in Pipe Sch 40	6.63	0.28					

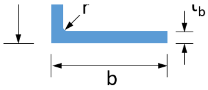


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30





ID	Name	d (in)	t_w (in)	b (in)	t_b (in)	r (in)		
34	L3x2x3/16	3.00	0.19	2.00	0.19	0.31		

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)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60
34	L3x2x3/16	0.92	0.01	0.17	0.98	0.01	0.33	0.82

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	23.56	23.56	11.22	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.15,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,2.03,1.16,1.18,1.18,1.17,1.17	300	200	1
4	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.70,1.68,1.67,1.67,1.65,1.70,1.67,1.67,1.67,1.67,1.68,1.68,1.53,1.70,1.67,1.67,1.66,1.71	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.63,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.18,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.15,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,2.03,1.16,1.18,1.18,1.17,1.17	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.63,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.18,1.65,1.67,1.67,1.66,1.66	300	200	1
8	20	3.75	3.75	3.75	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.31,2.32,2.32,2.32,2.32,2.32	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.70,1.68,1.67,1.67,1.65,1.70,1.67,1.67,1.67,1.67,1.68,1.68,1.53,1.70,1.67,1.67,1.66,1.71	300	200	1
11	20	3.75	3.75	3.75	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.29,2.32,2.32,2.32,2.32,2.32	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.70,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04,1.04,1.12,1.10,1.04,1.04,1.04,1.04	300	200	1
14	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.26,1.04,1.04,1.04,1.04,1.04,1.04,1.10,1.05,1.04,1.04,1.04,1.57	300	200	1
15	20	3.75	3.75	3.75	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.28,2.32,2.32,2.32,2.32,2.32	300	200	1
16	20	3.75	3.75	3.75	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.31,2.32,2.32,2.32,2.32,2.32	300	200	1
50	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
51	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
52	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
53	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	79.57	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	119.60	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	119.60	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.74	6.46	56.26	44.91
14	159.30	97.43	31.69	6.46	56.26	44.91
15	159.30	119.60	46.90	6.46	56.26	44.91
16	159.30	119.60	46.90	6.46	56.26	44.91
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15

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	0.238	0.622	0.000	0.023	0.000	0.645	#16	0.629	Not Required	Pass
2	0.004	0.587	0.074	0.124	0.011	0.663	#21	0.054	Not Required	Pass
3	0.005	0.831	0.098	0.083	0.039	0.932	#21	0.046	Not Required	Pass
4	0.005	0.813	0.043	0.081	0.007	0.859	#21	0.082	Not Required	Pass
5	0.005	0.515	0.014	0.082	0.002	0.532	#21	0.076	Not Required	Pass
6	0.005	0.831	0.098	0.083	0.039	0.932	#21	0.046	Not Required	Pass
7	0.005	0.516	0.014	0.082	0.002	0.532	#21	0.076	Not Required	Pass
8	0.005	0.087	0.023	0.039	0.007	0.106	#21	0.287	Not Required	Pass
9	0.016	0.083	0.034	0.001	0.000	0.125	#21	0.137	Not Required	Pass
10	0.005	0.813	0.043	0.081	0.007	0.859	#21	0.082	Not Required	Pass
11	0.004	0.089	0.028	0.040	0.008	0.114	#21	0.191	Not Required	Pass
12	0.004	0.587	0.074	0.124	0.011	0.663	#21	0.054	Not Required	Pass
13	0.004	0.307	0.051	0.058	0.010	0.351	#21	0.204	Not Required	Pass
14	0.009	0.307	0.063	0.057	0.010	0.344	#21	0.306	Not Required	Pass
15	0.004	0.089	0.028	0.040	0.008	0.114	#21	0.191	Not Required	Pass
16	0.005	0.087	0.023	0.039	0.007	0.106	#21	0.191	Not Required	Pass
50	0.103	0.010	0.003	0.002	0.001	0.114	#21	0.783	Not Required	Pass
51	0.020	0.008	0.012	0.002	0.001	0.037	#21	0.522	Not Required	Pass
52	0.103	0.010	0.003	0.002	0.001	0.114	#24	0.783	Not Required	Pass
53	0.020	0.008	0.012	0.002	0.001	0.037	#24	0.522	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

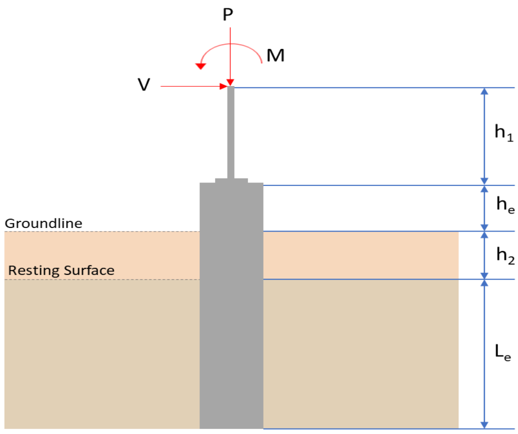
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 5.5$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ 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)	10.921	18.938
V_x (kip)	-1.019	-1.699
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	14.834	26.324

Material Properties

$f'_{ck} = 2.5$ 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,

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

$$H_o = \frac{(-1.019 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16226 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.834 \text{ kipft}) + ((-1.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.3621 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.175 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(5.175 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.175 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.175 \text{ ft})}{(5.5 \text{ ft})}$$

$$Ratio = 0.94091$$

Status: **PASS**
Ratio: **0.940**

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

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

$$q = \frac{(10.921 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.68256 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.68256 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.34128$$

Status: **PASS**
Ratio: **0.340**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.16226 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 2.3621 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3621 \text{ kipft/ft}) \times (5.5 \text{ ft})) + \left(3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft})^2\right)}{(6 \times (2.3621 \text{ kipft/ft})) + (4 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7589 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3621 \text{ kipft/ft})) + (3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.3621 \text{ kipft/ft})) + (2 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.21442 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.3621 \text{ kipft/ft})) + ((-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.76002 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7589 \text{ ft})}{2}$$

$$p_a = 0.28192 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.21442 \text{ kip/ft}^2\right)}{\left(0.28192 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.76057$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

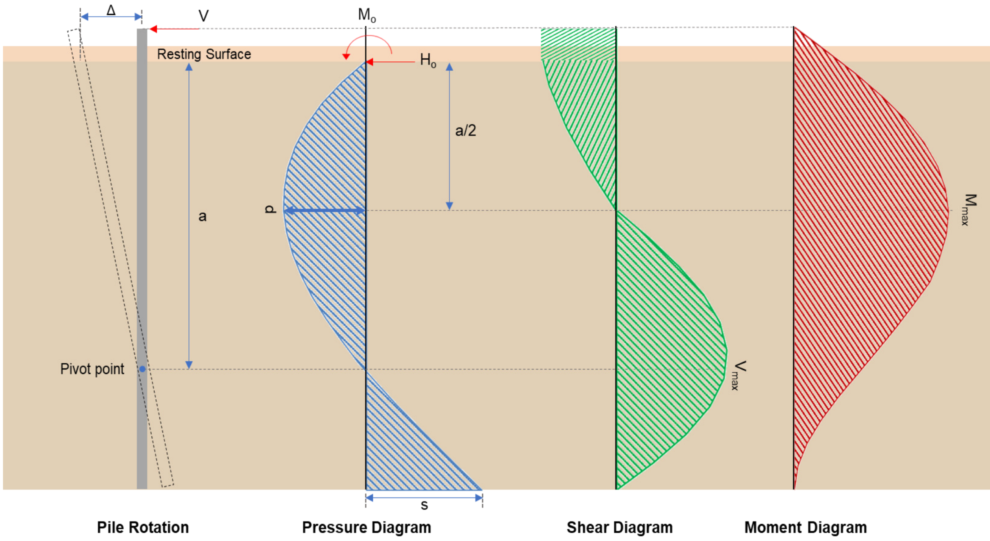
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.76002 \text{ kip/ft}^2\right)}{\left(0.825 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.92124$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.699 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27054 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.324 \text{ kipft}) + ((-1.699 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1917 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1917 \text{ kipft/ft})}{(-0.27054 \text{ kip/ft})}$$

$$E = 15.494 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1917 \text{ kipft/ft}) \times (5.5 \text{ ft})) + \left(3 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft})^2\right)}{(6 \times (4.1917 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7544 \text{ ft}$$

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

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

$$V_{max} = ((-0.27054 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.494 \text{ ft})}{(5.5 \text{ ft})} + 3\right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.494 \text{ ft})}{(5.5 \text{ ft})} + 2\right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})}\right)^3\right]\right]$$

V 6.1126 kip

$$V_{max} = 0.1120\ \text{kip}$$

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

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

$$M_{max} = ((-0.27054\ \text{kip/ft}) \times (48\ \text{in}) \times (5.5\ \text{ft})) \times \left[\left(\frac{(15.494\ \text{ft})}{(5.5\ \text{ft})} + \frac{(3.7544\ \text{ft})}{2 \times (5.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (15.494\ \text{ft})}{(5.5\ \text{ft})} + 3\right) \times \left(\frac{(3.7544\ \text{ft})}{2 \times (5.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (15.494\ \text{ft})}{(5.5\ \text{ft})} + 2\right) \times \left(\frac{(3.7544\ \text{ft})}{2 \times (5.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 16.266\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(18.938\ \text{kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2)\right)}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, \left(0.08 \times (2304\ \text{in}^2)\right)\right]$$

$$A_{st,required} = -83.967\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-83.967\ \text{in}^2\right), \left(0.0018 \times (2304\ \text{in}^2)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10⌀: 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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.938 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0070791$$

Status: **PASS**
Ratio: **0.010**

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 18.938 \text{ kip} \rightarrow 18938 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(18938\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 121.01\ \text{kip}$$

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

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

$$V_c = Min\left[(296.21\ \text{kip}), (121.01\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 121.01\ \text{kip}$$

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

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

$$\phi V_n = (0.65) \times ((121.01\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.74\ \text{kip}$$

Considering x-direction:

$V_{max} = 6.1126\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.1126\ \text{kip})}{(111.74\ \text{kip})}$$

$$Ratio = 0.054704$$

Status: **PASS**
Ratio: **0.050**

	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:** $M_{max} = 16.266\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(16.266\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.065168$$	Status: **PASS** Ratio: **0.070**