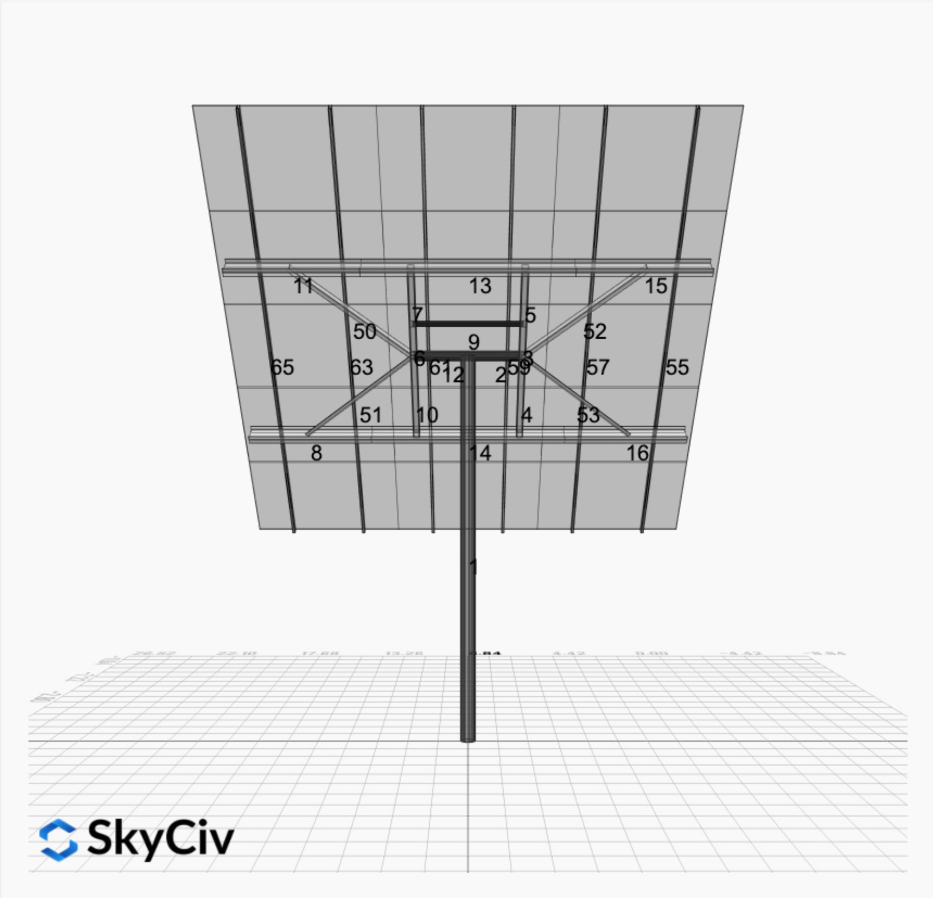
Project Details

Project Name: Pushak TOP15 - V1Jb
Location: 1888 Paradise Ridge Trail, Polebridge, MT 59928, USA
Unique ID: 1P-0-6TOP-HD-57-L-5Hx3W-STRUTS-65FH
Dealer: ______________________________

Date: Fri Oct 25 2024
Number of Modules: 15
Number of Poles: 1
Date Sold:
____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	17.68 ft
Winter Tilt Angle	50
Front Edge Clearance	7 ft

MT Solar Bill of Materials (1P-0-6TOP-HD-57-L-5Hx3W-STRUTS-65FH)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-57	57IN HD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	3

Rail Bill of Materials

Part	Qty
Rails (223in)	6
Rail Attachment	24
Module Mid Clamp	24
Module End Clamp	12

Part	Qty
Ground Lug	3

Site Details:

Site Address: 1888 Paradise Ridge Trail, Polebridge, MT 59928, USA

Array Specification

Duty Classification:	HD
Module Width:	44.64 in
Module Length:	69.73in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Winter Tilt Angle:	50
Front Edge Clearance:	7
Total Array Height at Tilt:	21.41 ft
Total Frame Length:	17.00 ft
Frame Weight:	1349 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	17.68 ft
Rail Length:	225.70 in
Rail Spacing:	2.95 ft

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	14.20 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: 6.75 ft
Foundation Volume:	4.000 y^3

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	1888 Paradise Ridge Trail, Polebridge, MT 59928, USA
Wind Speed:	99 mph
Snow Load:	103 psf

Design Disclaimer

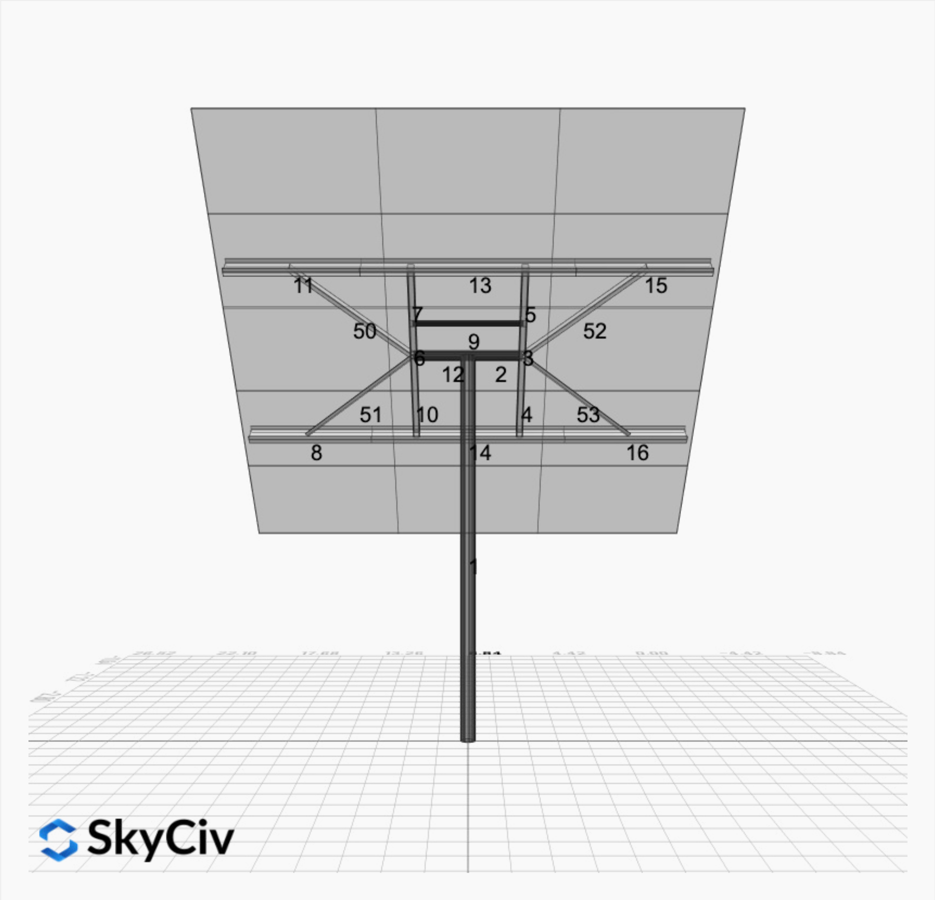
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

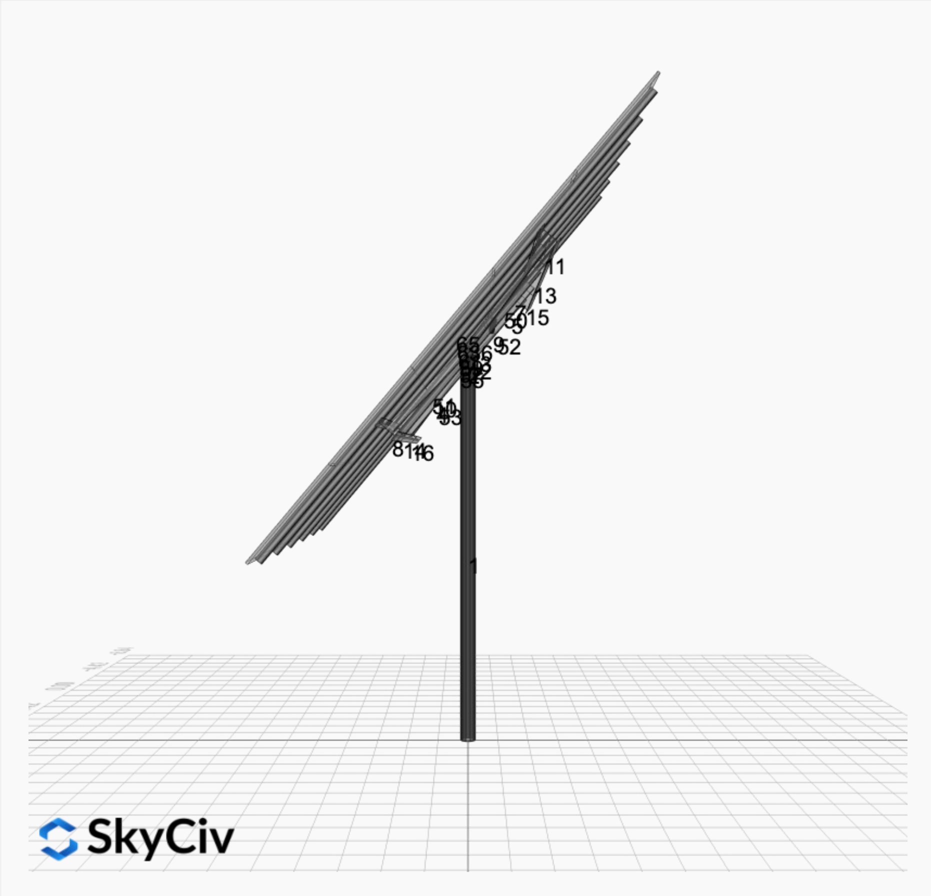
AutoDesigner Input

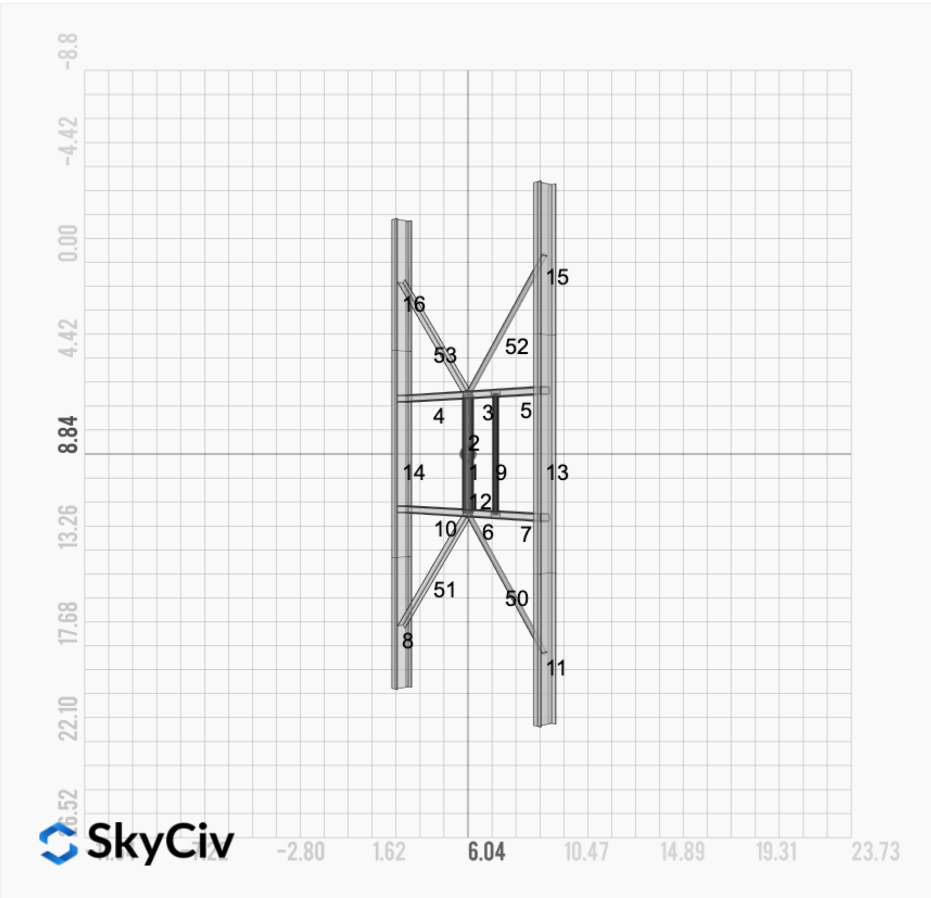
{"wind_speed_override":99,"snow_load_override":103,"direct_snow_load":false,"add_angle_brace":true,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"Pushak TOP15 - V1Jb","site_address":"1888 Paradise Ridge Trail, Polebridge, MT 59928, USA","module_width":44.64,"module_length":69.73,"number_rows":"5","number_columns":3,"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":50,"ground_clearance":7,"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":false}

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)




SkyCiv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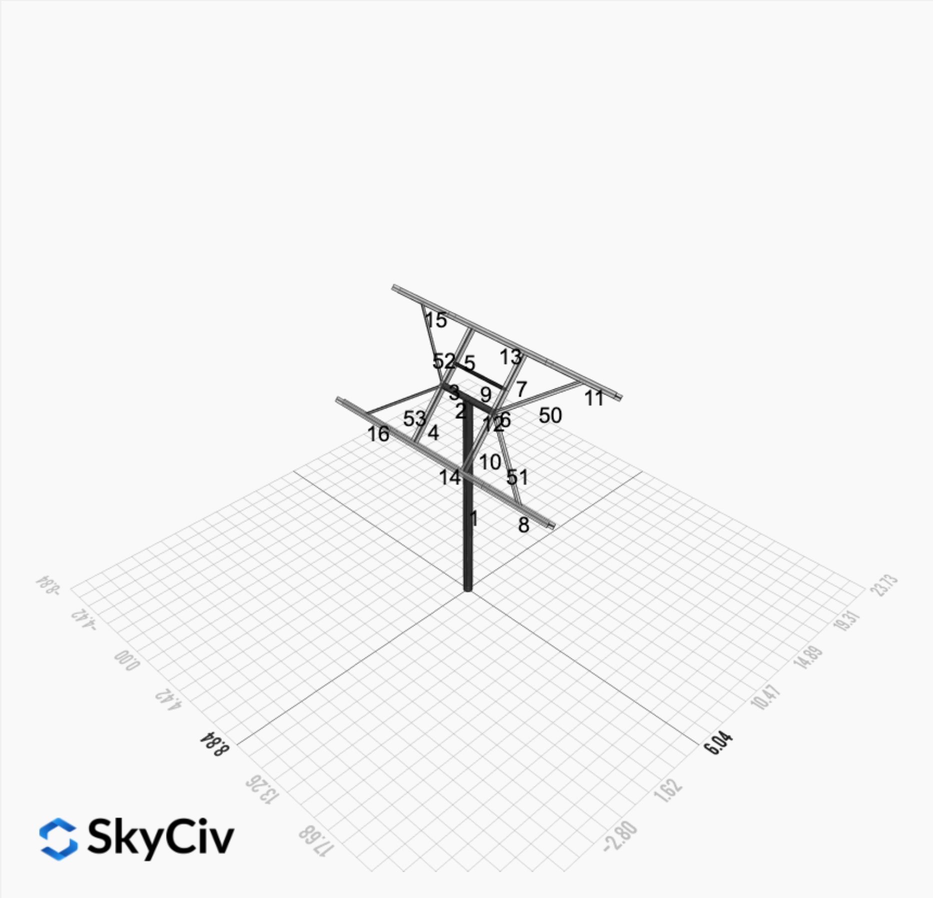
0.00
4.42
8.84
13.26
17.68
22.10
16
15
53
52
4
3
5
2
14
1
9
13
12
10
6
7
51
50
8
11
-2.80
1.62
6.04
10.47
14.89
19.31
23.73
SkyCiv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5
13
9
7
11
50
16
53
4
2
12
6
10
14
51
1
8
-8.84
-4.42
0.00
4.42
8.84
13.26
17.68
23.73
19.31
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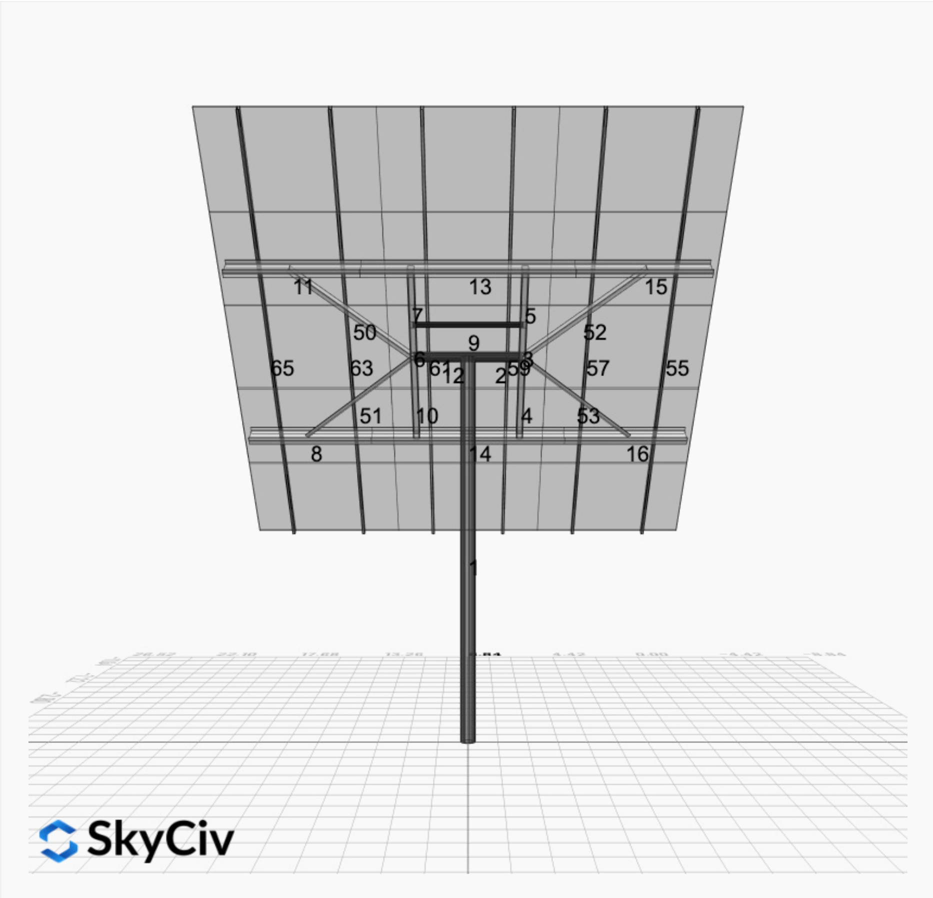
10.47
6.04
1.62
-2.80
SkyCiv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52
9
6
3
65
63
61
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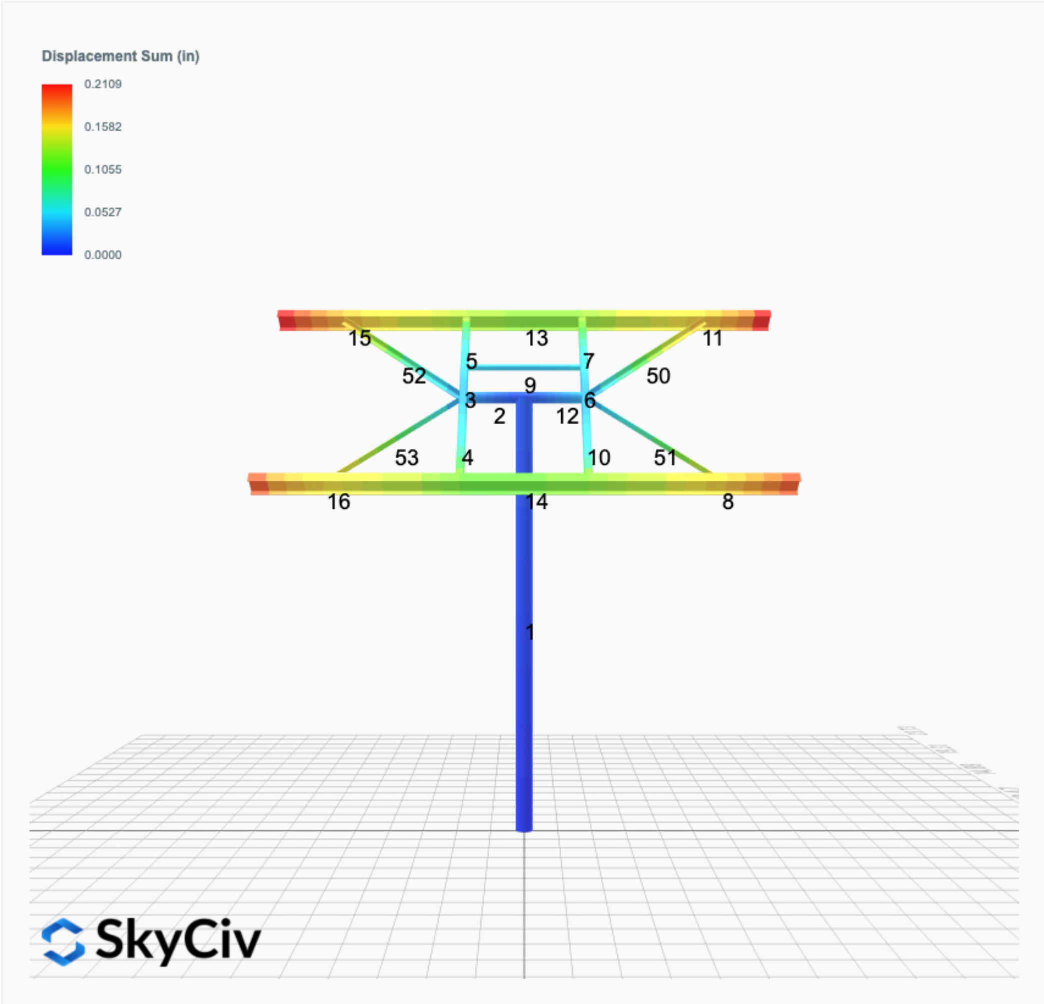
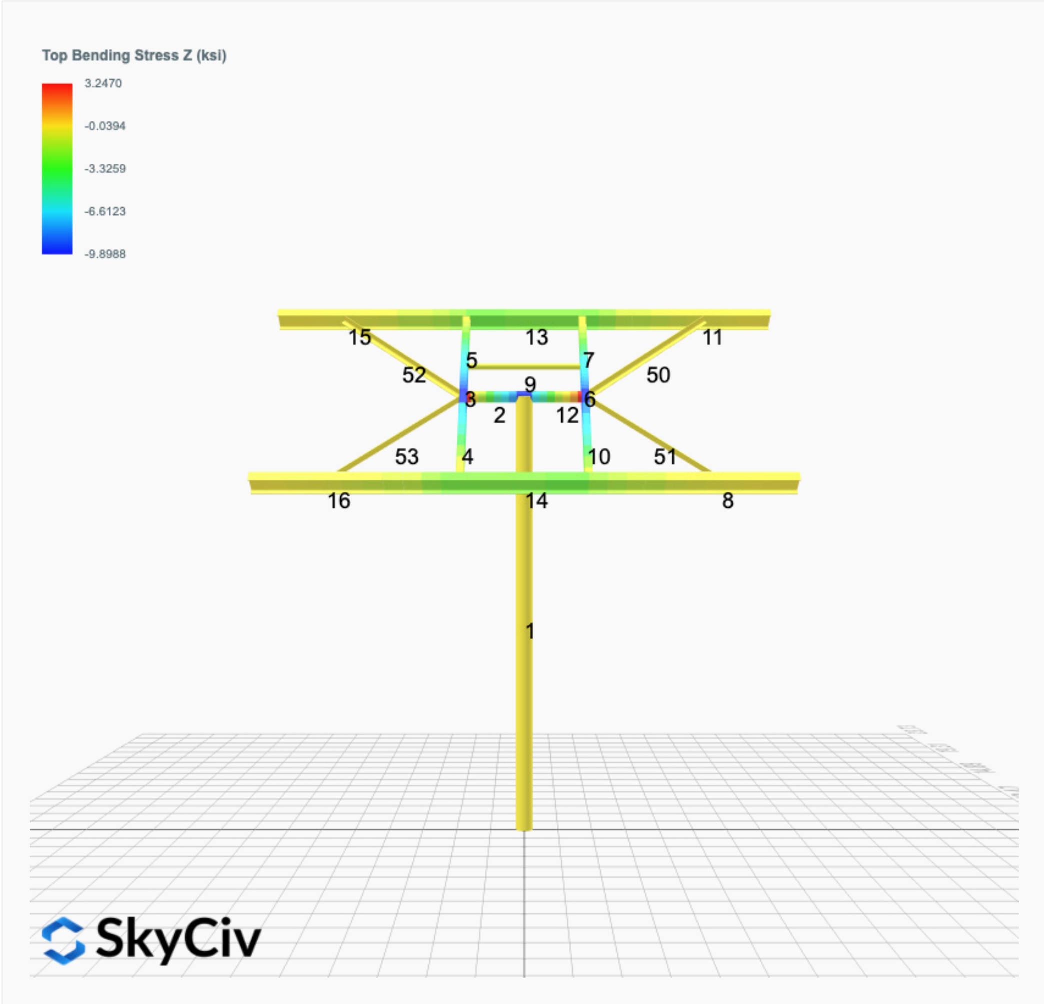
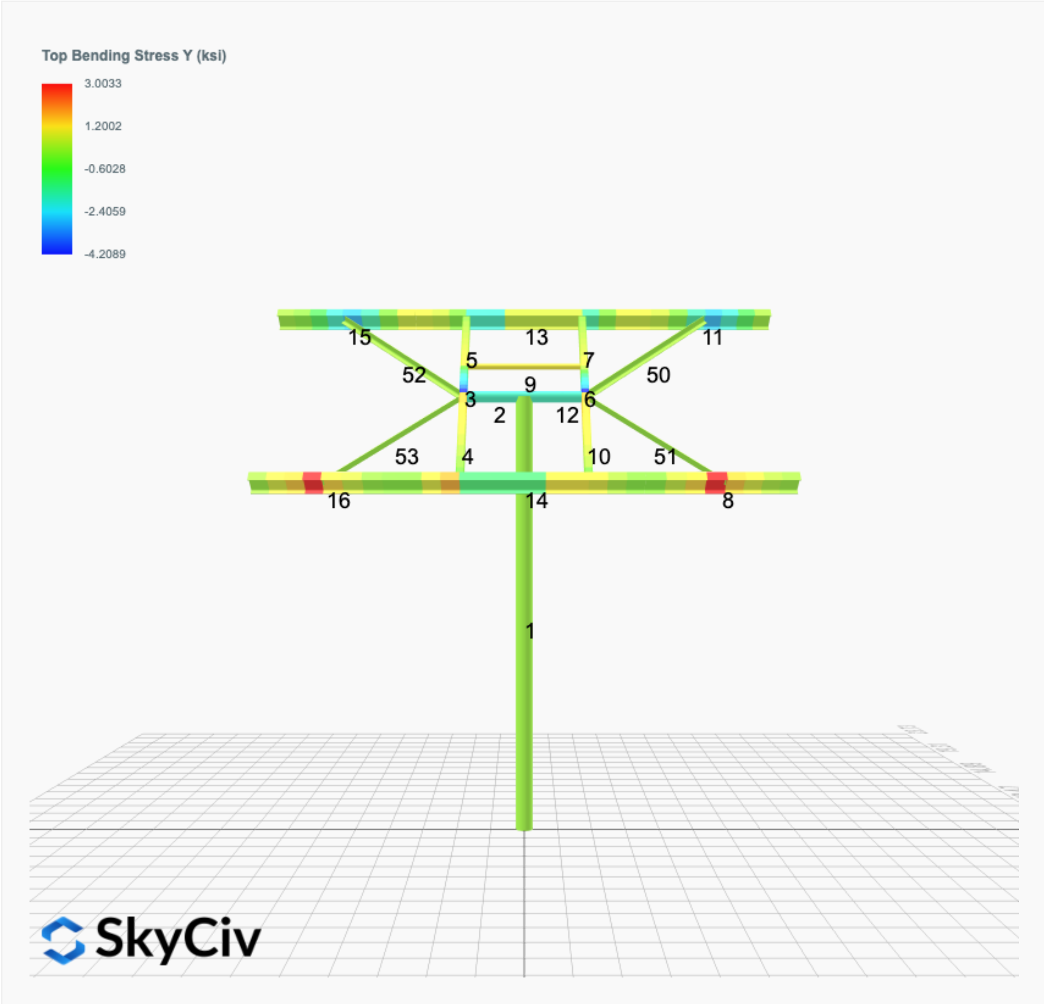
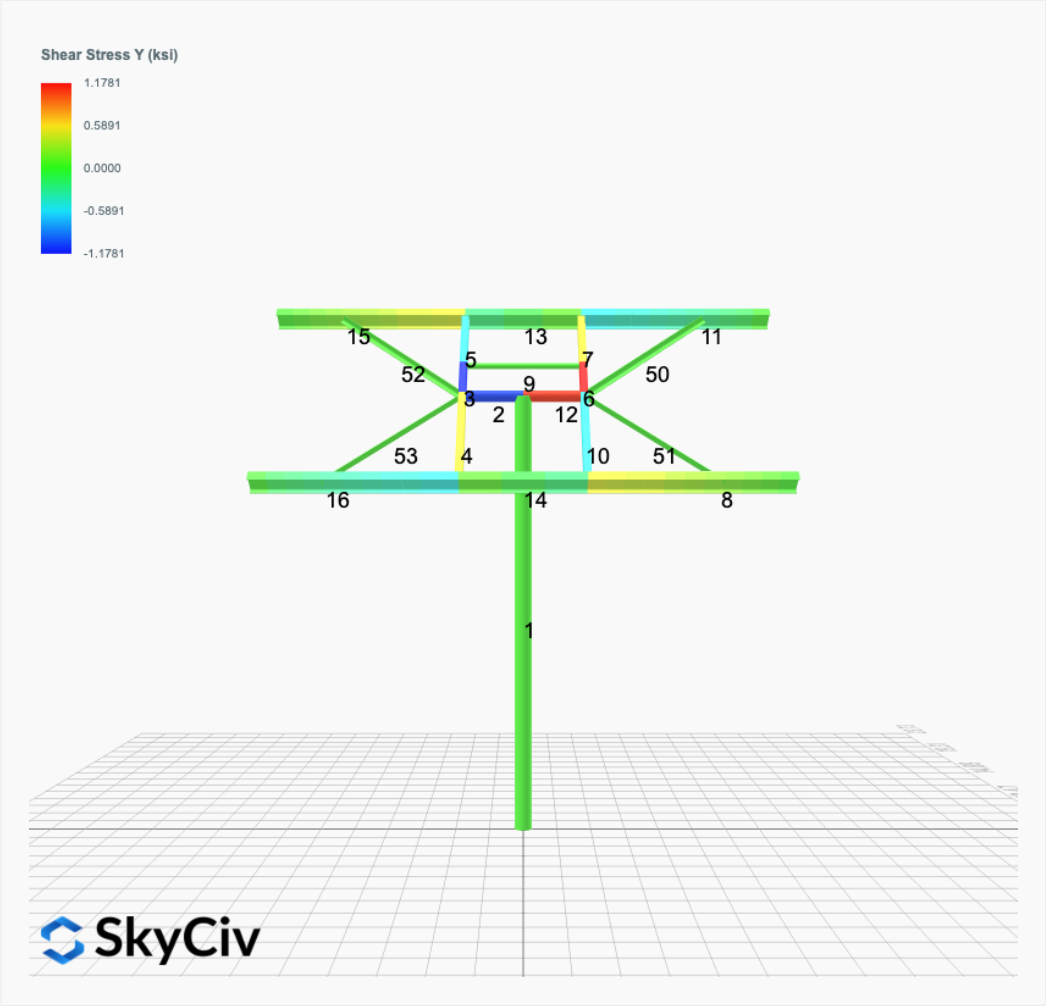
2
59
57
55
51
10
4
53
8
14
16
1
SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
3.2470
-0.0394
-3.3259
-6.6123
-9.8988
15
13
11
5
7
52
50
9
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Top Bending Stress Y (ksi)
3.0033
1.2002
-0.6028
-2.4059
-4.2089
15
13
11
5
7
52
9
50
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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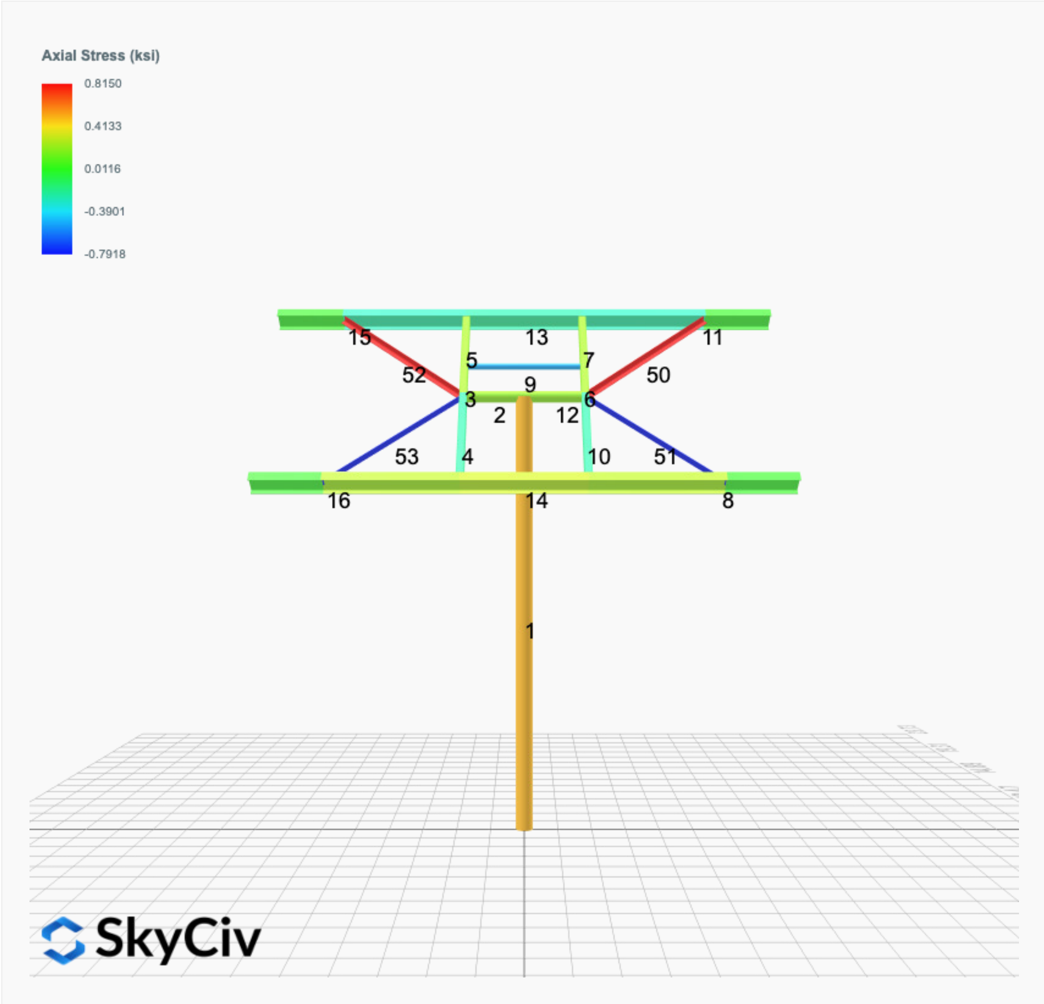
0.5891
0.0000
-0.5891
-1.1781
15
13
11
5
7
52
50
9
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.8150
0.4133
0.0116
-0.3901
-0.7918
15
13
11
5
7
52
9
50
3
6
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	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 2. D + L	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 3. D + (S or Lr or R)	0.0000	7.1972	0.0000	-0.0000	-0.0000	0.0604
ULS: 3. D + (S or Lr or R)	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.0333	0.0000	-0.0000	-0.0000	0.0510
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 5b. D + 0.7E	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.0333	0.0000	-0.0000	-0.0000	0.0510
ULS: 8. 0.6D + 0.7E	0.0000	1.5249	0.0000	-0.0000	-0.0000	0.0136
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9562	4.1830	0.0000	-0.0000	-0.0000	28.4525
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.9562	0.9001	0.0000	-0.0000	-0.0000	-27.1525
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4671	7.2644	0.0000	-0.0000	-0.0000	21.3734
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	6.0333	0.0000	-0.0000	-0.0000	0.0510
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4671	4.8022	0.0000	-0.0000	-0.0000	-20.3304
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	6.0333	0.0000	-0.0000	-0.0000	0.0510
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4671	3.7726	0.0000	-0.0000	-0.0000	21.3451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4671	1.3105	0.0000	-0.0000	-0.0000	-20.3587
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.5415	0.0000	-0.0000	-0.0000	0.0227
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9562	3.1663	0.0000	-0.0000	-0.0000	28.4435
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.5249	0.0000	-0.0000	-0.0000	0.0136
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.9562	-0.1165	0.0000	-0.0000	-0.0000	-27.1615
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.5249	0.0000	-0.0000	-0.0000	0.0136

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	11.8668
Shear X	-3.2603
Shear Z	0.0000
Moment X	-0.0024
Moment Y (Twist)	0.0017
Moment Z	49.8020

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	7.2644
Shear X	-1.9562
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	28.4525

Project Details

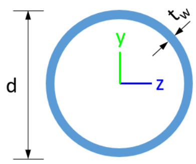
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

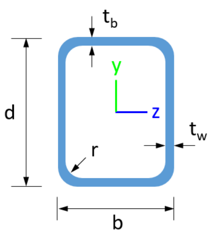
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

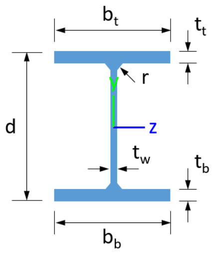
Section Dimensions



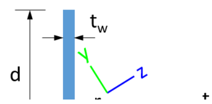
ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
8	6in Pipe Sch 80	6.63	0.43					

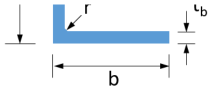


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



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





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)
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
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
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
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	8	29.83	29.83	14.20	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.18,1.19	300	200	1
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.63,1.68,1.67,1.68,1.66,1.68	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.64,1.68,1.67,1.68,1.66,1.68	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.18,1.19	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
8	19	4.75	4.75	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.32,2.35,2.32,2.33,2.33,2.32,2.33,2.33,2.32,2.30,2.32,2.33,2.33,2.31,2.33,2.32,2.33,2.32,2.33	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.63,1.68,1.67,1.68,1.66,1.68	300	200	1
11	19	4.75	4.75	4.75	2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.35,2.33,2.33,2.33,2.32,2.33,2.33,2.32,2.30,2.32,2.33,2.33,2.31,2.33,2.33,2.33,2.32,2.33	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
14	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
15	19	4.75	4.75	4.75	2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.35,2.33,2.33,2.33,2.32,2.33,2.33,2.32,2.30,2.32,2.33,2.33,2.31,2.33,2.33,2.33,2.32,2.33	300	200	1
16	19	4.75	4.75	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.32,2.35,2.32,2.33,2.33,2.32,2.33,2.33,2.32,2.30,2.32,2.33,2.33,2.31,2.33,2.32,2.33,2.32,2.33	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 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	378.22	71.40	62.23	62.23	113.47	113.47
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	95.15	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	95.15	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	23.58	6.12	40.24	43.62
14	133.20	85.85	23.57	6.12	40.24	43.62
15	133.20	95.15	32.87	6.12	40.24	43.62
16	133.20	95.15	32.87	6.12	40.24	43.62
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	0.166	0.800	0.000	0.029	0.000	0.857	#13	0.815	Not Required	Pass
2	0.007	0.398	0.162	0.096	0.027	0.527	#21	0.053	Not Required	Pass
3	0.007	0.515	0.174	0.051	0.069	0.692	#21	0.045	Not Required	Pass
4	0.008	0.507	0.071	0.051	0.011	0.582	#21	0.080	Not Required	Pass
5	0.007	0.319	0.028	0.051	0.007	0.351	#21	0.074	Not Required	Pass
6	0.007	0.515	0.174	0.051	0.069	0.692	#21	0.045	Not Required	Pass
7	0.007	0.319	0.028	0.051	0.007	0.352	#21	0.074	Not Required	Pass
8	0.012	0.086	0.090	0.030	0.012	0.115	#21	0.339	Not Required	Pass
9	0.026	0.035	0.050	0.001	0.000	0.098	#21	0.136	Not Required	Pass
10	0.008	0.507	0.071	0.051	0.011	0.582	#21	0.080	Not Required	Pass
11	0.009	0.087	0.090	0.030	0.012	0.113	#21	0.226	Not Required	Pass
12	0.007	0.398	0.162	0.096	0.027	0.527	#21	0.053	Not Required	Pass
13	0.009	0.237	0.050	0.041	0.010	0.291	#21	0.190	Not Required	Pass
14	0.016	0.239	0.060	0.040	0.010	0.291	#21	0.286	Not Required	Pass
15	0.009	0.087	0.090	0.030	0.012	0.113	#21	0.226	Not Required	Pass
16	0.012	0.086	0.090	0.030	0.012	0.115	#21	0.226	Not Required	Pass
50	0.186	0.009	0.005	0.003	0.002	0.198	#21	0.783	Not Required	Pass
51	0.037	0.004	0.015	0.002	0.002	0.051	#21	0.522	Not Required	Pass
52	0.186	0.009	0.005	0.003	0.002	0.198	#23	0.783	Not Required	Pass
53	0.037	0.004	0.015	0.002	0.002	0.051	#23	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/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

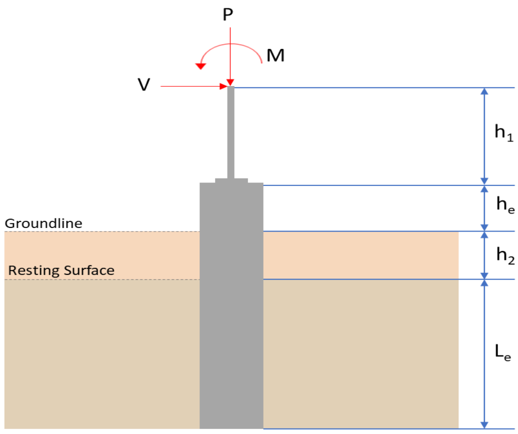
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48\ \mathrm{in}$ - Pile width

$D = 48\ \mathrm{in}$ - Pile depth

$L = 6.75\ \mathrm{ft}$ - 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)	7.264	11.867
V_x (kip)	-1.956	-3.260
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.002
M_z (kipft)	28.453	49.802

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.956\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.31146\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.453 \text{ kipft}) + ((-1.956 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.5307 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.2612 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2612 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.261 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.261 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.92756$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(7.264 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.454 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.454 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.227$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.31146\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.5307\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.5307\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.31146\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (4.5307\ \mathrm{kipft/ft})) + (4 \times (-0.31146\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6329\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.5307\ \mathrm{kipft/ft})) + (3 \times (-0.31146\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^2}{(6.75\ \mathrm{ft})^2 \times [(3 \times (4.5307\ \mathrm{kipft/ft})) + (2 \times (-0.31146\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.24481\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.5307\ \mathrm{kipft/ft})) + ((-0.31146\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.91642\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6329\ \mathrm{ft})}{2}$$

$$p_a = 0.34747\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.24481\ \mathrm{kip/ft^2}\right)}{\left(0.34747\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.70456$$

Status: **PASS**
Ratio: **0.700**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.75\ \mathrm{ft})$$

$$p_s = 1.0125\ \mathrm{kip/ft^2}$$

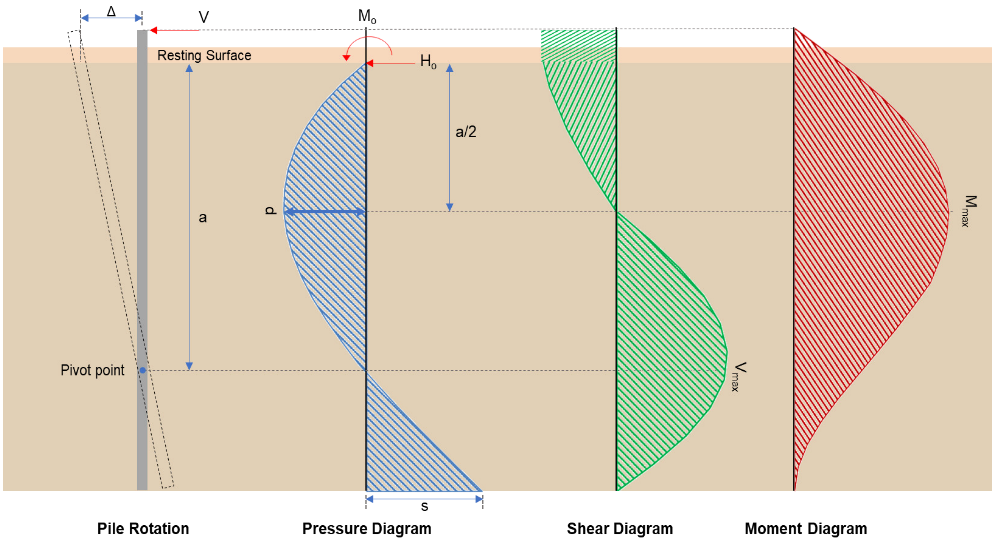
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.91642\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.90511$$

Status: PASS
Ratio: 0.910



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.26\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.51911\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.802\text{ kipft}) + ((-3.26\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 7.9303\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.9303\text{ kipft/ft})}{(-0.51911\text{ kip/ft})}$$

$$E = 15.277\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9303\text{ kipft/ft}) \times (6.75\text{ ft})) + \left(3 \times (-0.51911\text{ kip/ft}) \times (6.75\text{ ft})^2\right)}{(6 \times (7.9303\text{ kipft/ft})) + (4 \times (-0.51911\text{ kip/ft}) \times (6.75\text{ ft}))}$$

$$a = 4.628\text{ ft}$$

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

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

$$V_{max} = ((-0.51911\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.277\text{ ft})}{(6.75\text{ ft})} + 3\right) \times \left(\frac{(4.628\text{ ft})}{(6.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.277\text{ ft})}{(6.75\text{ ft})} + 2\right) \times \left(\frac{(4.628\text{ ft})}{(6.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.6884\text{ kip}$$

$$V_{max} = 9.0884 \text{ kip}$$

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

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

$$M_{max} = ((-0.51911 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(15.277 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.628 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.277 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.628 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.277 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.628 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 31.421 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57 \, b}$$

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z \, H)}{1.57 \, b}$$

$$M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00031847 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

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

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

$$V_{max} = 12 \left(\frac{M_o \, b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(6.75 \text{ ft})} \right) \times \left(\frac{(4.5 \text{ ft})}{(6.75 \text{ ft})} - 1 \right) \times \left(\frac{(4.5 \text{ ft})}{(6.75 \text{ ft})} \right)^2$$

$$V_{max} = 0.00033551 \text{ kip}$$

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

$$M_{max} = (M_o \, b) \left[1 - \left(4 \, \frac{a}{2 \, L_e}^3 \right) + \left(3 \, \frac{a}{2 \, L_e}^4 \right) \right]$$

$$M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.5 \text{ ft})}{2 \times (6.75 \text{ ft})}^3 \right) + \left(3 \times \frac{(4.5 \text{ ft})}{2 \times (6.75 \text{ ft})}^4 \right) \right]$$

$$M_{max} = 0.0011323 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.867\ \text{kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5\ \text{ksi}) \times \left(2304\ \text{in}^2\right)\right)}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, \left(0.08 \times \left(2304\ \text{in}^2\right)\right)\right]$$

$$A_{st,required} = -84.202\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.202\ \text{in}^2\right), \left(0.0018 \times \left(2304\ \text{in}^2\right)\right)\right]$$

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

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,

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

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

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

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

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

$$Ratio = \frac{(11.867 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.004436$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.867 \text{ kip} \rightarrow 11867 \text{ lbf}$,
22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11867 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.07 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.07 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.07 \text{ kip}$$

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 $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.07 \text{ kip}) + (50.894 \text{ kip}))$$ $$\phi V_n = 111.12 \text{ kip}$$ **Considering x-direction:** $V_{max} = 9.6884 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(9.6884 \text{ kip})}{(111.12 \text{ kip})}$$ $$Ratio = 0.087185$$	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:** $M_{max} = 31.421\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(31.421\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.12589$$	Status: **PASS** Ratio: **0.130**
	Considering z-direction: $M_{max} = 0.0011323\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.0011323\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 4.5366 \times 10^{-6}$$	Status: **PASS** Ratio: **0.000**