Project Details

Project Name: Adrian Marti - V1Jb

Location: 2280 Rhyolite Dr, Jefferson, CO 80456, USA

Unique ID: 1P-0-8TOP-SD-84-L-4Hx3W-STRUTS-E933

Dealer: ______________________________

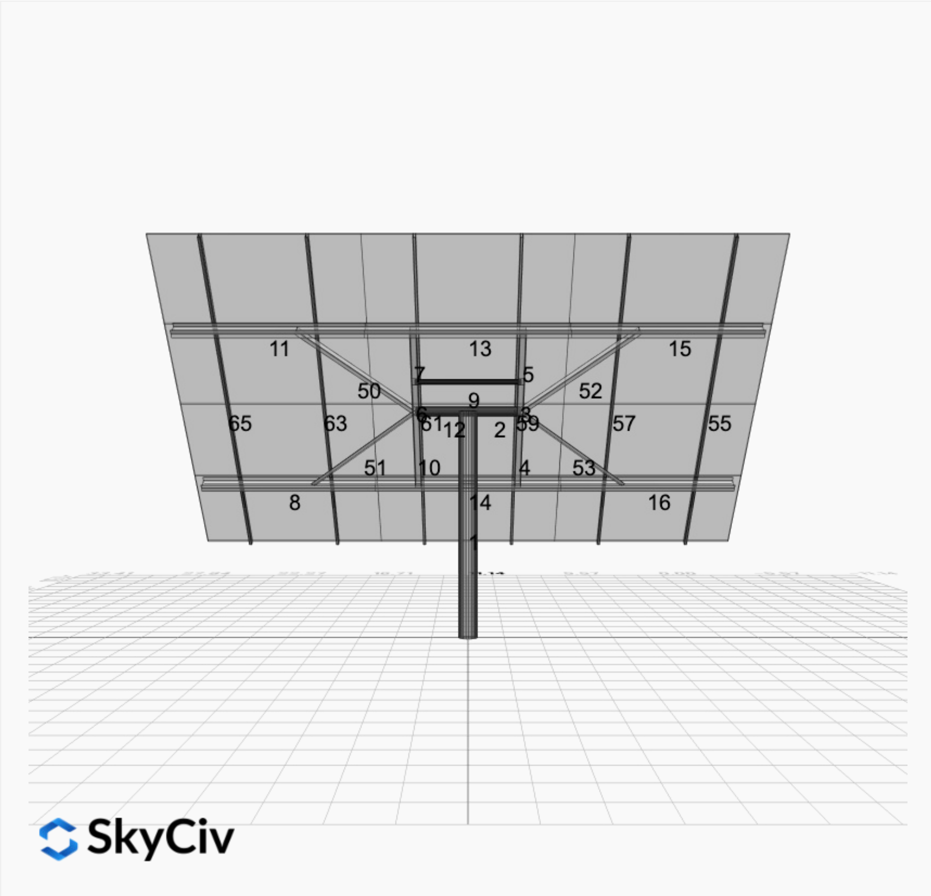
Date: Wed Nov 27 2024

Number of Modules: 12

Number of Poles: 1

Date Sold:

____



Array Dimensions N/S	14.87 ft
Array Dimensions E/W	22.27 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

MT Solar Bill of Materials (1P-0-8TOP-SD-84-L-4Hx3W-STRUTS-E933)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-84	84IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (176in)	6
Rail Attachment	12
Module Mid Clamp	18

Part	Qty
Module End Clamp	12
Ground Lug	3

Site Details:

Site Address: 2280 Rhyolite Dr, Jefferson, CO 80456, USA

Array Specification

Duty Classification:	SD
Module Width:	44.10 in
Module Length:	88.10in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	50
Front Edge Clearance:	3
Total Array Height at Tilt:	14.39 ft
Total Frame Length:	21.50 ft
Frame Weight:	1111 lbs
Array Dimensions N/S:	14.87 ft
Array Dimensions E/W:	22.27 ft
Rail Length:	178.40 in
Rail Spacing:	3.71 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.69 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: 5.75 ft
Foundation Volume:	3.407 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2280 Rhyolite Dr, Jefferson, CO 80456, USA
Wind Speed:	105 mph
Snow Load:	81 psf

Design Disclaimer

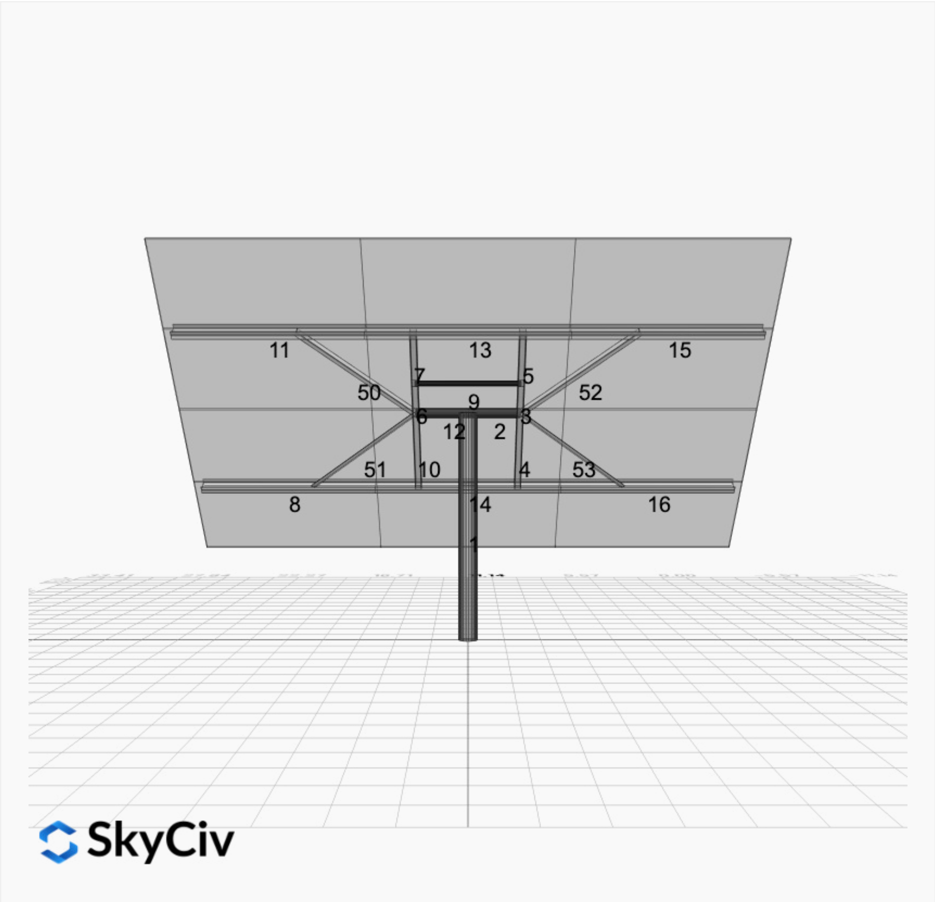
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

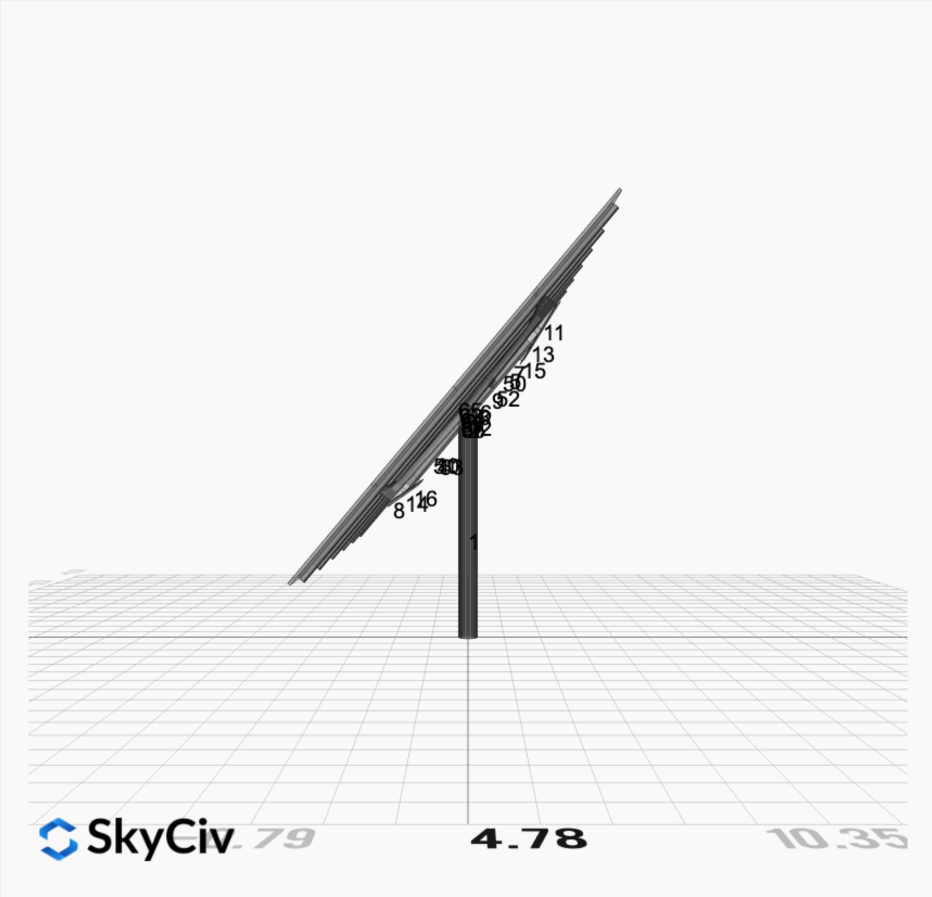
AutoDesigner Input

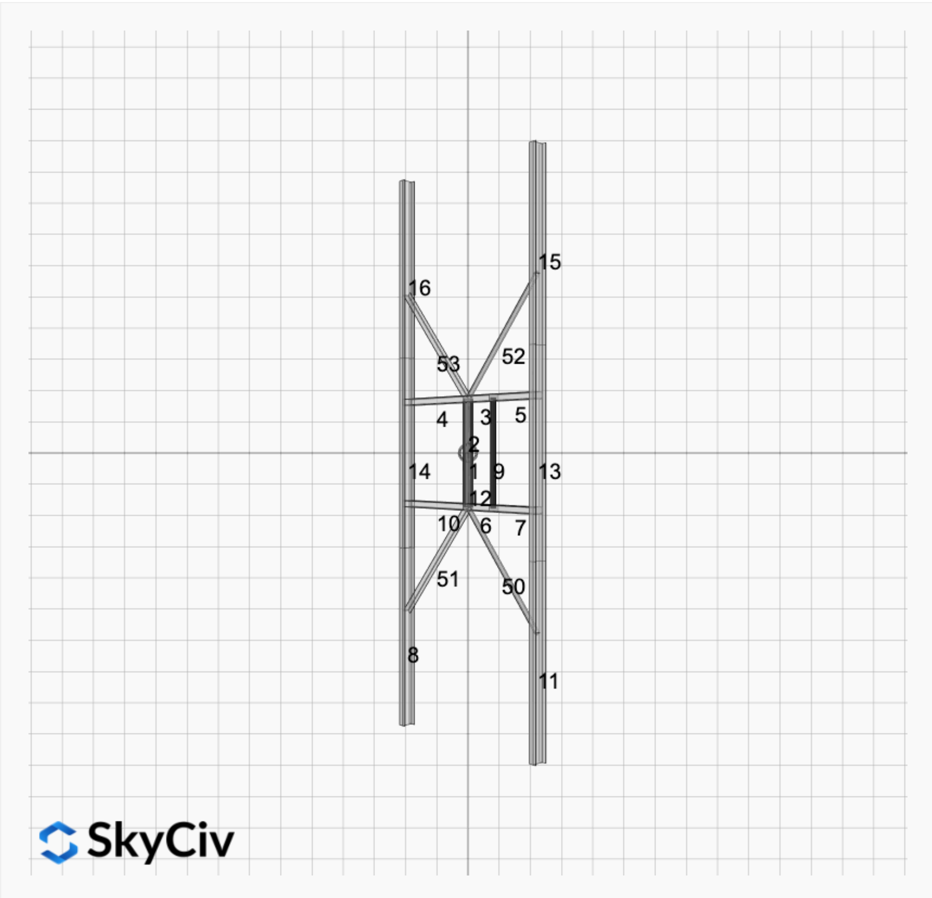

```
{"product_type":"Beam","designer_name":"Andrew Luehring","designer_email":"owner@co-reliableelectric.com","designer_phone":"9706586222","project_id":"Adrian Marti - V1Jb","site_address":"2280 Rhyolite Dr, Jefferson, CO 80456, USA","module_width":44.1,"module_length":88.1,"number_rows":"4","number_columns":3,"pole_mount_section":"4_40","core_pipe_width":65,"core_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":3,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"8_40","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false,"wind_speed_override":105,"snow_load_override":81,"direct_snow_load":false,"add_angle_brace":true}
```

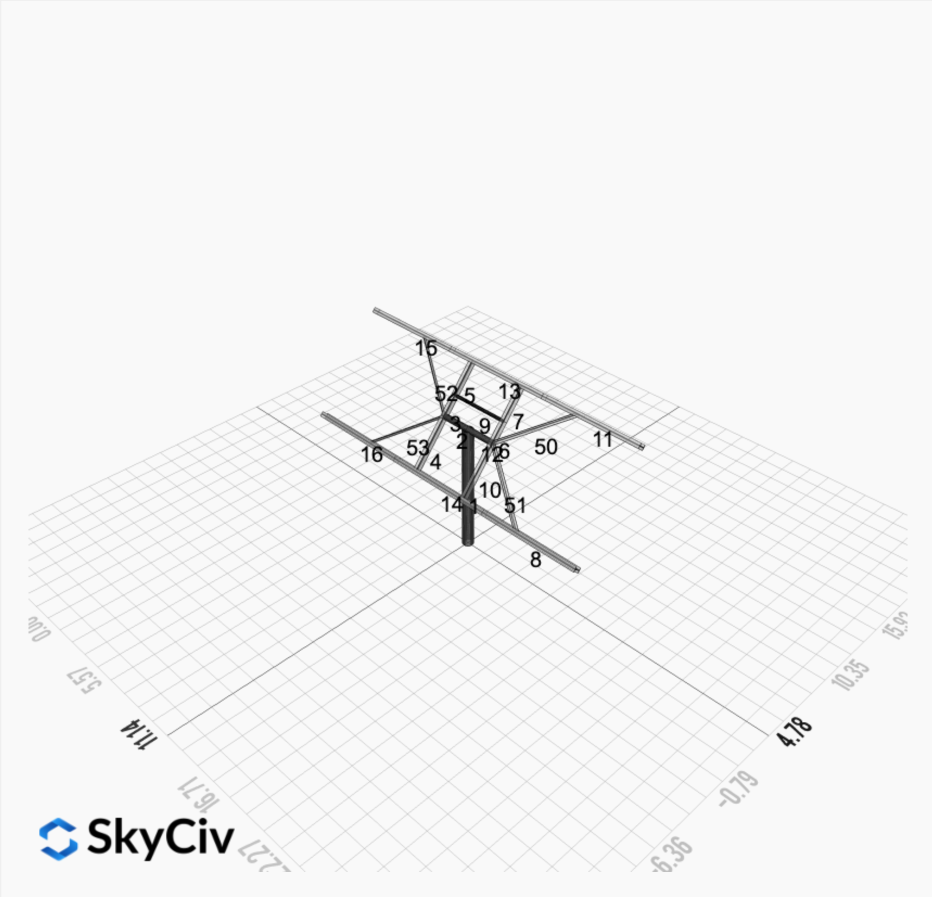

Design Notes:

- 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)

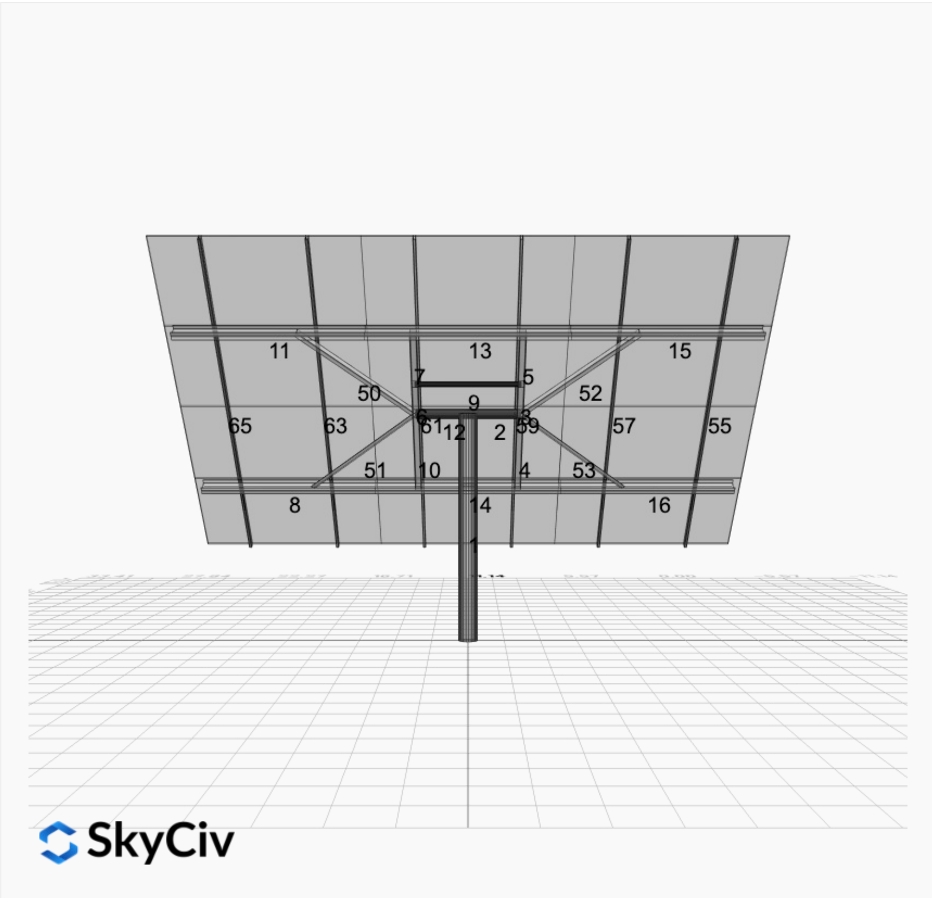



11
13
15
52
8
14
16
1
4.78
10.35


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

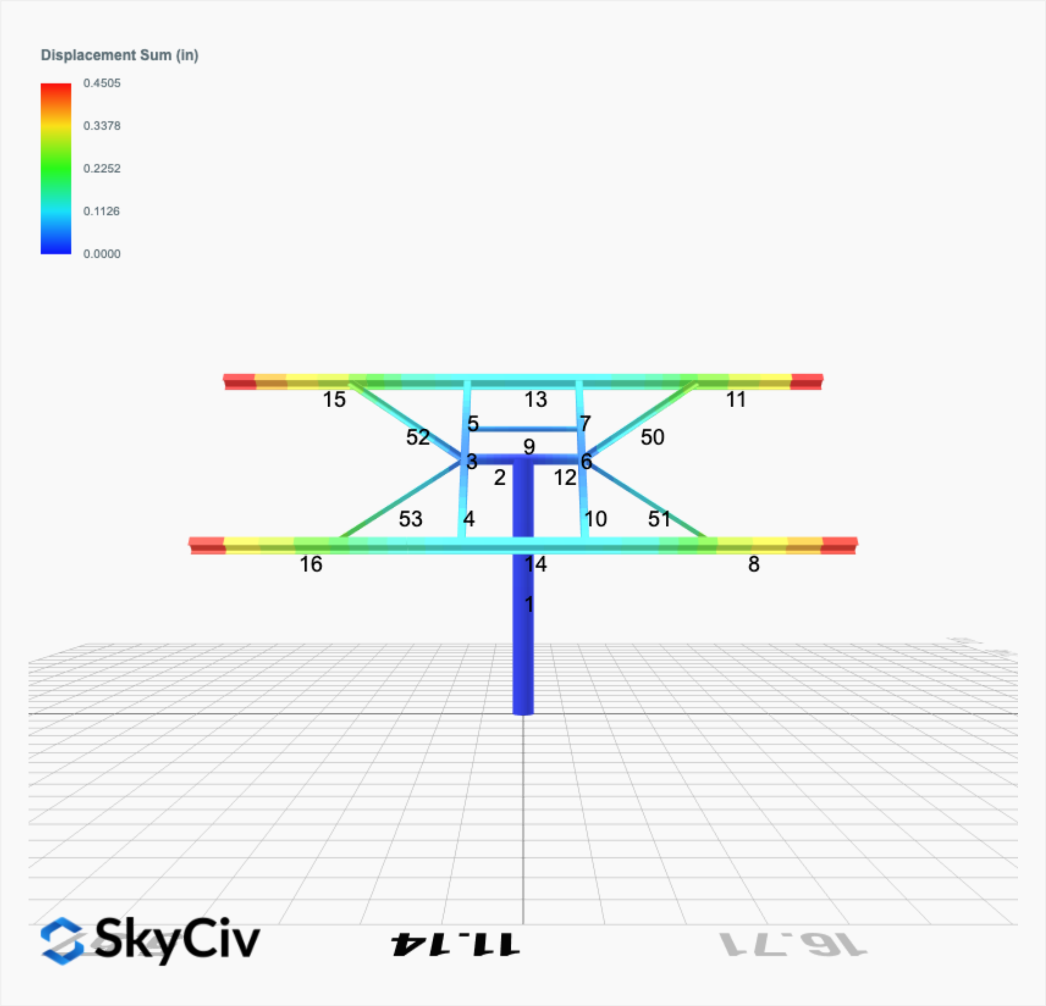


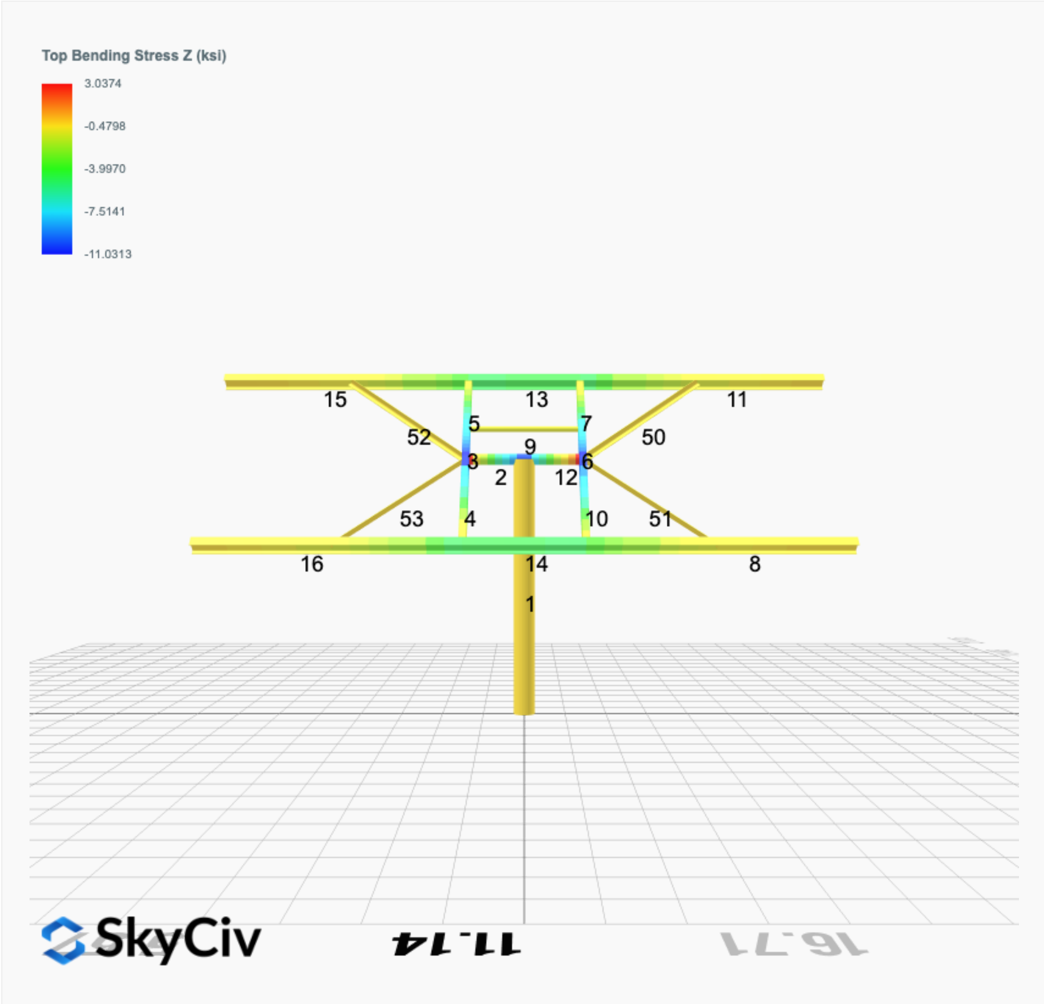
15
52
5
13
7
9
3
2
12
6
50
11
16
53
4
10
14
1
51
8
0.00
5.57
11.14
16.71
22.27
15.92
10.35
4.78
-0.79
-6.36
SkyCiv

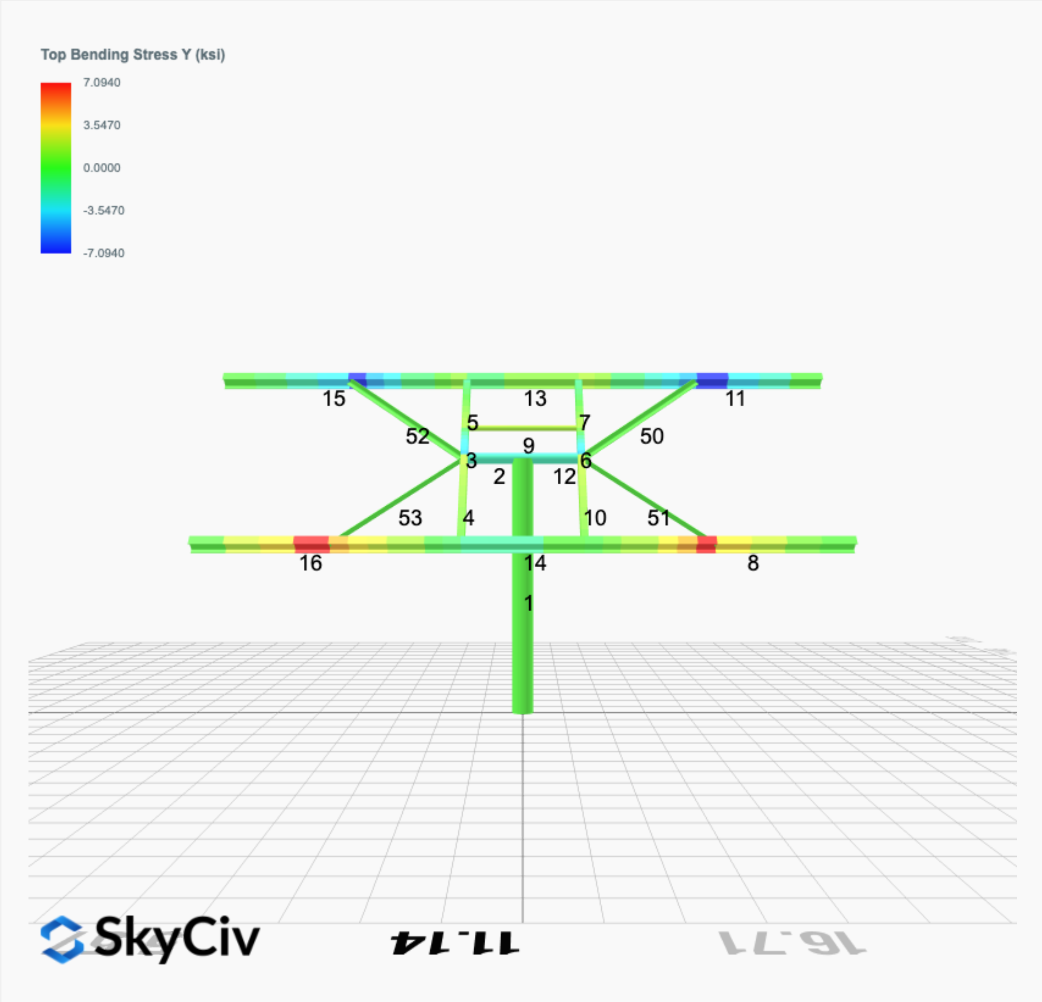


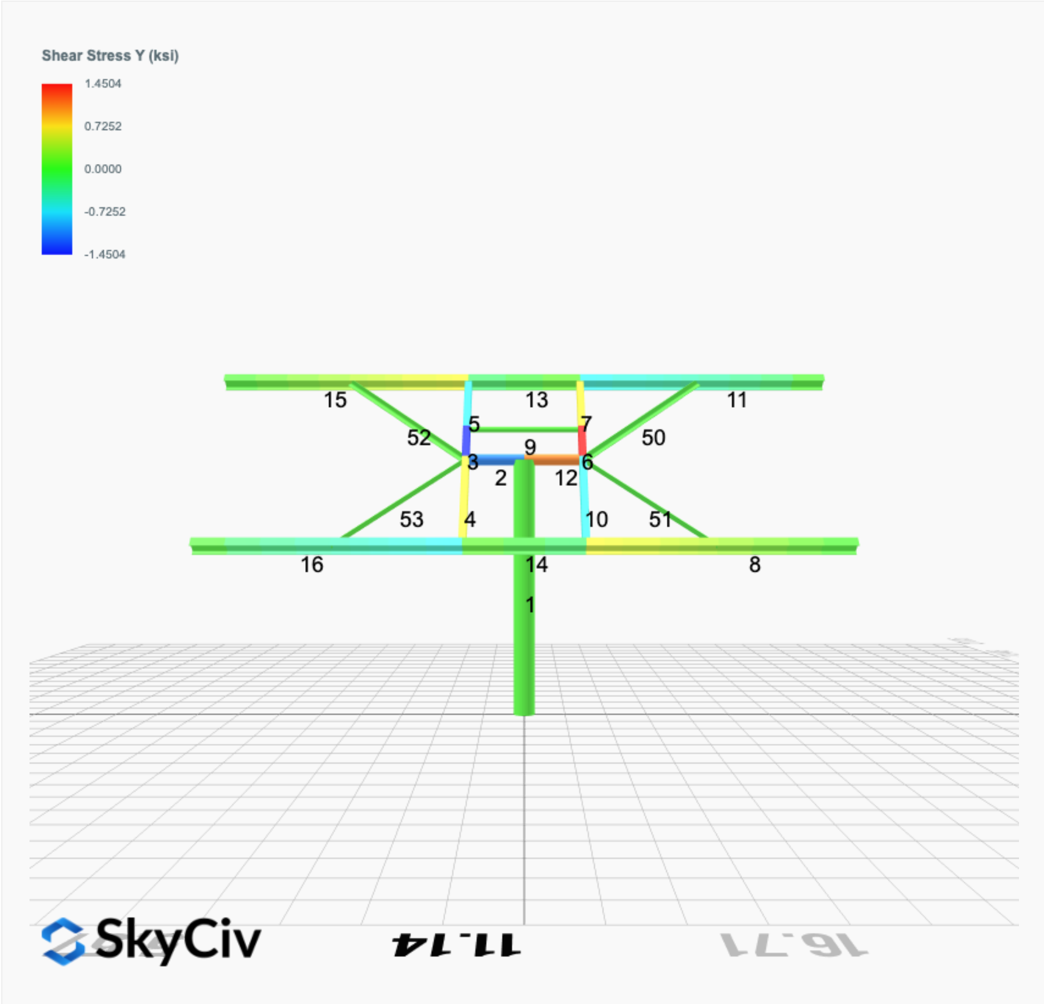
11
13
15
7
5
50
52
9
65
63
6
61
12
2
3
59
57
55
51
10
4
53
8
14
16
1
SkyCiv

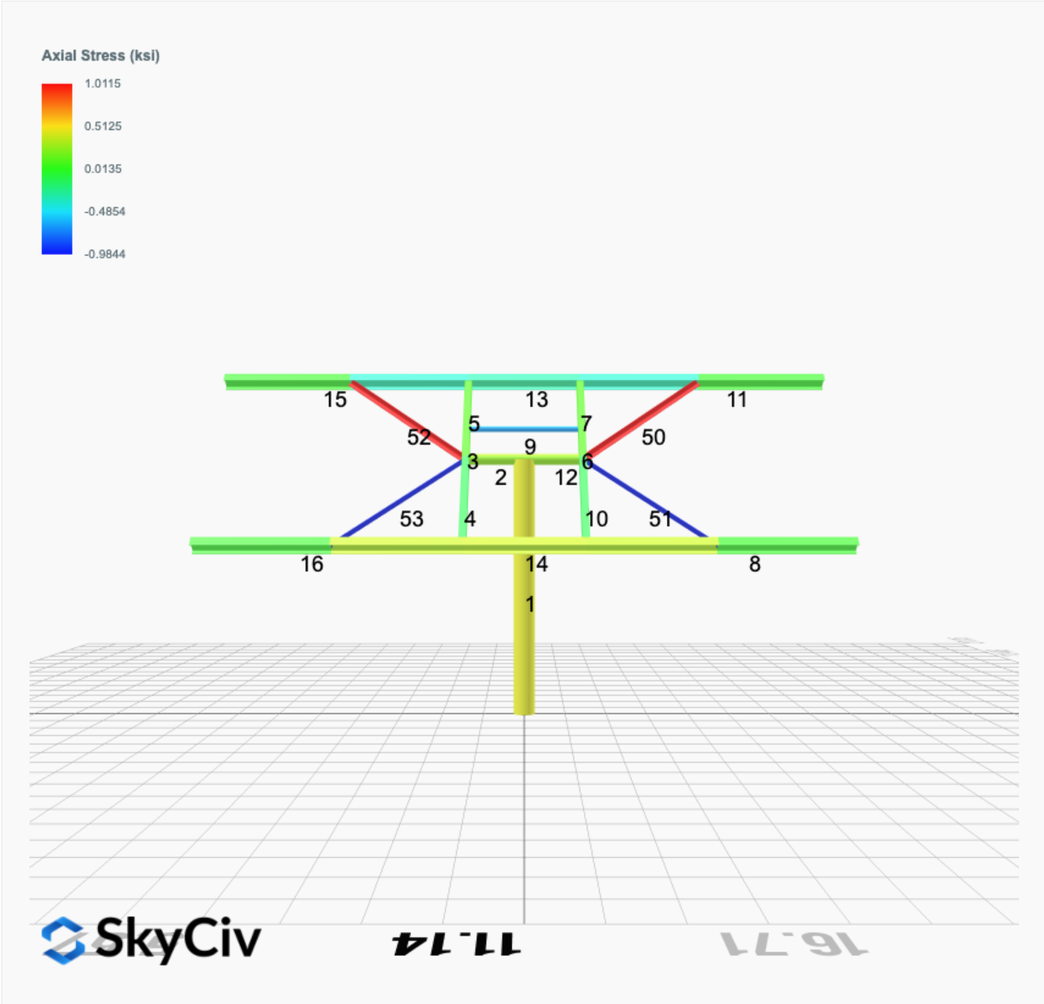
FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
3.0374
-0.4798
-3.9970
-7.5141
-11.0313
15
13
11
5
7
52
9
50
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Top Bending Stress Y (ksi)
7.0940
3.5470
0.0000
-3.5470
-7.0940
15
13
11
5
7
52
50
9
3
6
2
12
53
4
10
51
16
14
8
1


Shear Stress Y (ksi)
1.4504
0.7252
0.0000
-0.7252
-1.4504
15
13
11
5
7
52
9
50
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Axial Stress (ksi)
1.0115
0.5125
0.0135
-0.4854
-0.9844
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	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 2. D + L	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 3. D + (S or Lr or R)	0.0000	6.0242	0.0000	-0.0000	-0.0000	0.0689
ULS: 3. D + (S or Lr or R)	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	5.1092	0.0000	-0.0000	-0.0000	0.0571
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 5b. D + 0.7E	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	5.1092	0.0000	-0.0000	-0.0000	0.0571
ULS: 8. 0.6D + 0.7E	0.0000	1.4185	0.0000	-0.0000	-0.0000	0.0132
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2329	4.2378	0.0000	-0.0000	-0.0000	19.5406
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2329	0.4905	0.0000	-0.0000	-0.0000	-19.2875
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6747	6.5144	0.0000	-0.0000	-0.0000	14.6962
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	5.1092	0.0000	-0.0000	-0.0000	0.0571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6747	3.7040	0.0000	-0.0000	-0.0000	-14.4249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	5.1092	0.0000	-0.0000	-0.0000	0.0571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6747	3.7694	0.0000	-0.0000	-0.0000	14.6610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6747	0.9589	0.0000	-0.0000	-0.0000	-14.4601
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.3642	0.0000	-0.0000	-0.0000	0.0219
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2329	3.2921	0.0000	-0.0000	-0.0000	19.5319
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.4185	0.0000	-0.0000	-0.0000	0.0132
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2329	-0.4551	0.0000	-0.0000	-0.0000	-19.2962
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.4185	0.0000	-0.0000	-0.0000	0.0132

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	10.2544
Shear X	-3.7215
Shear Z	0.0000
Moment X	-0.0009
Moment Y (Twist)	0.0004
Moment Z	32.9886

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	6.5144
Shear X	-2.2329
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	19.5406

Project Details

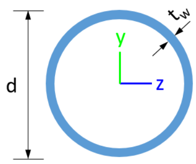
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

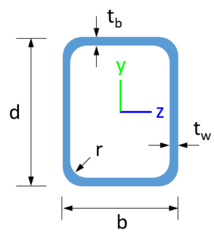
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

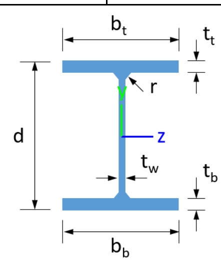
Section Dimensions



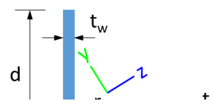
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
9	8in Pipe Sch 40	8.63	0.32					

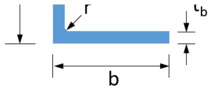


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25





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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23
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	9	18.26	18.26	8.69	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
8	18	7.00	7.00	7.00	2.33,2.32,2.33,2.31,2.32,2.33,2.32,2.32,2.31,2.32,2.32,2.33,2.31,2.33,2.31,2.31,2.31,2.31,2.32,2.33,2.29,2.33,2.32,2.33,2.31,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.31,2.32,2.32,2.33,2.31,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.29,2.33,2.32,2.33,2.31,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	18	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
15	18	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.31,2.32,2.32,2.33,2.31,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.29,2.33,2.32,2.33,2.31,2.33	300	200	1
16	18	7.00	7.00	7.00	2.33,2.32,2.33,2.31,2.32,2.33,2.32,2.32,2.31,2.32,2.32,2.33,2.31,2.33,2.31,2.31,2.31,2.31,2.32,2.33,2.29,2.33,2.32,2.33,2.31,2.33	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	377.97	251.67	83.29	83.29	113.39	113.39
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	64.33	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	6.26	29.14	16.61
11	120.60	64.33	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.03	17.90	6.45	30.09	45.74
14	120.60	84.03	17.90	6.45	30.09	45.74
15	120.60	64.33	23.36	6.45	30.09	45.74
16	120.60	64.33	23.36	6.45	30.09	45.74
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	0.041	0.396	0.000	0.033	0.000	0.412	#13	0.373	Not Required	Pass
2	0.010	0.495	0.259	0.116	0.043	0.685	#21	0.052	Not Required	Pass
3	0.003	0.670	0.266	0.068	0.087	0.938	#21	0.044	Not Required	Pass
4	0.003	0.654	0.088	0.066	0.007	0.743	#21	0.078	Not Required	Pass
5	0.003	0.416	0.060	0.067	0.019	0.476	#21	0.073	Not Required	Pass
6	0.003	0.670	0.266	0.068	0.087	0.938	#21	0.044	Not Required	Pass
7	0.003	0.416	0.060	0.067	0.019	0.476	#21	0.073	Not Required	Pass
8	0.024	0.187	0.228	0.041	0.015	0.316	#21	0.464	Not Required	Pass
9	0.036	0.059	0.078	0.001	0.000	0.155	#21	0.132	Not Required	Pass
10	0.003	0.654	0.088	0.066	0.007	0.743	#21	0.078	Not Required	Pass
11	0.013	0.189	0.228	0.043	0.016	0.311	#21	0.309	Not Required	Pass
12	0.010	0.495	0.259	0.116	0.043	0.685	#21	0.052	Not Required	Pass
13	0.013	0.386	0.039	0.053	0.008	0.431	#21	0.177	Not Required	Pass
14	0.019	0.387	0.039	0.052	0.007	0.419	#21	0.265	Not Required	Pass
15	0.013	0.189	0.228	0.043	0.016	0.311	#21	0.309	Not Required	Pass
16	0.024	0.187	0.228	0.041	0.015	0.316	#21	0.309	Not Required	Pass
50	0.246	0.009	0.005	0.002	0.001	0.258	#21	0.783	Not Required	Pass
51	0.049	0.004	0.014	0.001	0.001	0.063	#6	0.522	Not Required	Pass
52	0.246	0.009	0.005	0.002	0.001	0.258	#23	0.783	Not Required	Pass
53	0.049	0.004	0.014	0.001	0.001	0.063	#6	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

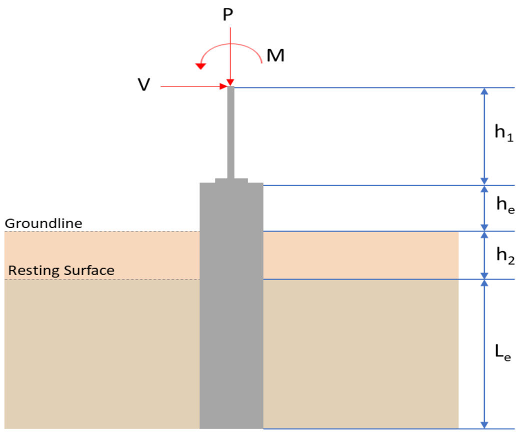
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 5.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.514	10.254
V_x (kip)	-2.233	-3.721
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	19.541	32.989

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.233\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.35557\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.541\text{ kipft}) + ((-2.233\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 3.1116\text{ kipft/ft}$$

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:
$L_{e,x} = 5.1744\text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(5.1744\text{ ft}), (0\text{ ft})]$$

$$L_{e,req} = 5.174\text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75\text{ ft}) - (0\text{ ft}) - (0\text{ ft})$$

$$L_e = 5.75\text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.174\text{ ft})}{(5.75\text{ ft})}$$

$$Ratio = 0.89983$$

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

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

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

$$q = \frac{(6.514\text{ kip})}{(16\text{ ft}^2)}$$

$$q = 0.40712\text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.40712\text{ kip/ft}^2)}{(2000\text{ psf})}$$

$$Ratio = 0.20356$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.35557\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.1116\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1116\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (-0.35557\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (3.1116\ \mathrm{kipft/ft})) + (4 \times (-0.35557\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 3.9793\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1116\ \mathrm{kipft/ft})) + (3 \times (-0.35557\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]^2}{(5.75\ \mathrm{ft})^2 \times [(3 \times (3.1116\ \mathrm{kipft/ft})) + (2 \times (-0.35557\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}$$

$$p = 0.17233\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.1116\ \mathrm{kipft/ft})) + ((-0.35557\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}{(5.75\ \mathrm{ft})^2}$$

$$s = 0.75833\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.9793\ \mathrm{ft})}{2}$$

$$p_a = 0.29845\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.17233\ \mathrm{kip/ft^2}\right)}{\left(0.29845\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.57743$$

Status: **PASS**
Ratio: **0.580**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 0.8625\ \mathrm{kip/ft^2}$$

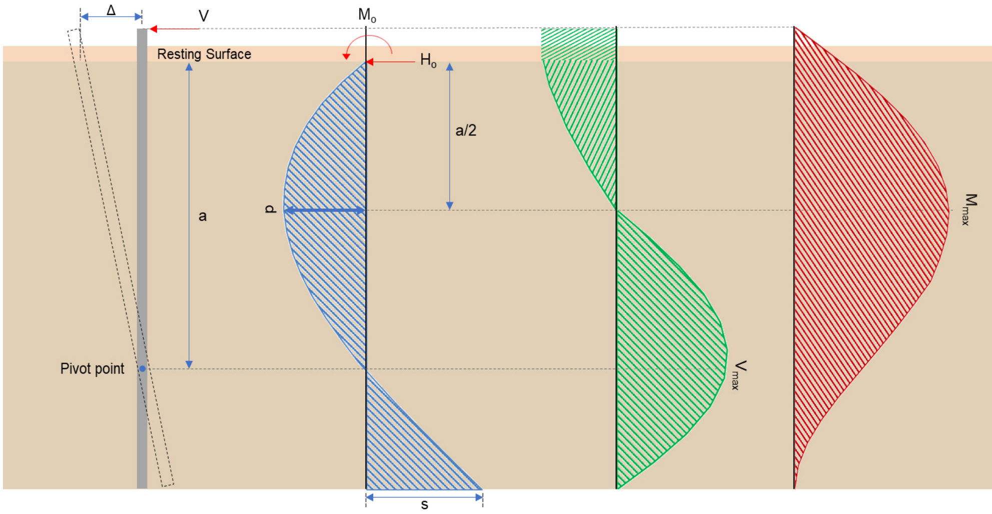
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.75833\ \mathrm{kip/ft^2}\right)}{\left(0.8625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.87922$$

Status: **PASS**
Ratio: **0.880**



Pile Rotation **Pressure Diagram** **Shear Diagram** **Moment Diagram**

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.721\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.59252\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.989\ \text{kipft}) + ((-3.721\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 5.253\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.253\ \text{kipft/ft})}{(-0.59252\ \text{kip/ft})}$$

$$E = 8.8656\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.253\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.59252\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (5.253\ \text{kipft/ft})) + (4 \times (-0.59252\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.978\ \text{ft}$$

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

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

$$V_{max} = ((-0.59252\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8656\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.978\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (8.8656\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.978\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.020\ \text{kip}$$

$$V_{max} = 8.029\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.59252\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (5.75\ \mathrm{ft})) \times \left[\left(\frac{(8.8656\ \mathrm{ft})}{(5.75\ \mathrm{ft})} + \frac{(3.978\ \mathrm{ft})}{2 \times (5.75\ \mathrm{ft})} \right) - \left[\left(\frac{4 \times (8.8656\ \mathrm{ft})}{(5.75\ \mathrm{ft})} + 3 \right) \times \left(\frac{(3.978\ \mathrm{ft})}{2 \times (5.75\ \mathrm{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8656\ \mathrm{ft})}{(5.75\ \mathrm{ft})} + 2 \right) \times \left(\frac{(3.978\ \mathrm{ft})}{2 \times (5.75\ \mathrm{ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.848\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001\ \mathrm{kipft}) + ((0\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.00015924\ \mathrm{kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (0\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (0.00015924\ \mathrm{kipft/ft})) + (4 \times (0\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 3.8333\ \mathrm{ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00015924\ \mathrm{kipft/ft}) \times (48\ \mathrm{in})}{(5.75\ \mathrm{ft})} \right) \times \left(\frac{(3.8333\ \mathrm{ft})}{(5.75\ \mathrm{ft})} - 1 \right) \times \left(\frac{(3.8333\ \mathrm{ft})}{(5.75\ \mathrm{ft})} \right)^2$$

$$V_{max} = 0.00019693\ \mathrm{kip}$$

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

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

$$M_{max} = ((0.00015924\ \mathrm{kipft/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left(4 \times \frac{(3.8333\ \mathrm{ft})}{2 \times (5.75\ \mathrm{ft})}^3 \right) + \left(3 \times \frac{(3.8333\ \mathrm{ft})}{2 \times (5.75\ \mathrm{ft})}^4 \right) \right]$$

$$M_{max} = 0.00056617\ \mathrm{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(10.254\text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5\text{ ksi}) \times \left(2304\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85 \times (2.5\text{ ksi}))}, \left(0.08 \times \left(2304\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.255\text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.255\text{ in}^2\right), \left(0.0018 \times \left(2304\text{ in}^2\right)\right)\right]$$

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

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**

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

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

$$Ratio = \frac{(10.254\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.003833$$

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 $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}, P = 10.254\ \mathrm{kip} \rightarrow 10254\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(10254\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 119.85\ \mathrm{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (119.85\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 119.85\ \mathrm{kip}$$

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

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

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

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

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

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

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

$$\phi V_n = (0.65) \times ((119.85 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.99 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.029 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(8.029 \text{ kip})}{(110.99 \text{ kip})}$$

$$Ratio = 0.072343$$

Status: **PASS**
Ratio: **0.070**

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, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\ \mathrm{kipft}), (2121.6\ \mathrm{kipft})]$$ $$\phi M_n = 249.6\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 21.848\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(21.848\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.087532$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $M_{max} = 0.00056617\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.00056617\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 2.2683 \times 10^{-6}$$	Status: **PASS** Ratio: **0.000**