Project Details

Project Name: Jonathan Cook - V2Jb

Location: Waldron, WA 98245, USA

Unique ID: 1P-0-8TOP-XD-84-L-4Hx4W-STRUTS-GI7K

Dealer: ______________________________

Date: Tue Mar 11 2025

Number of Modules: 16

Number of Poles: 1

Date Sold:

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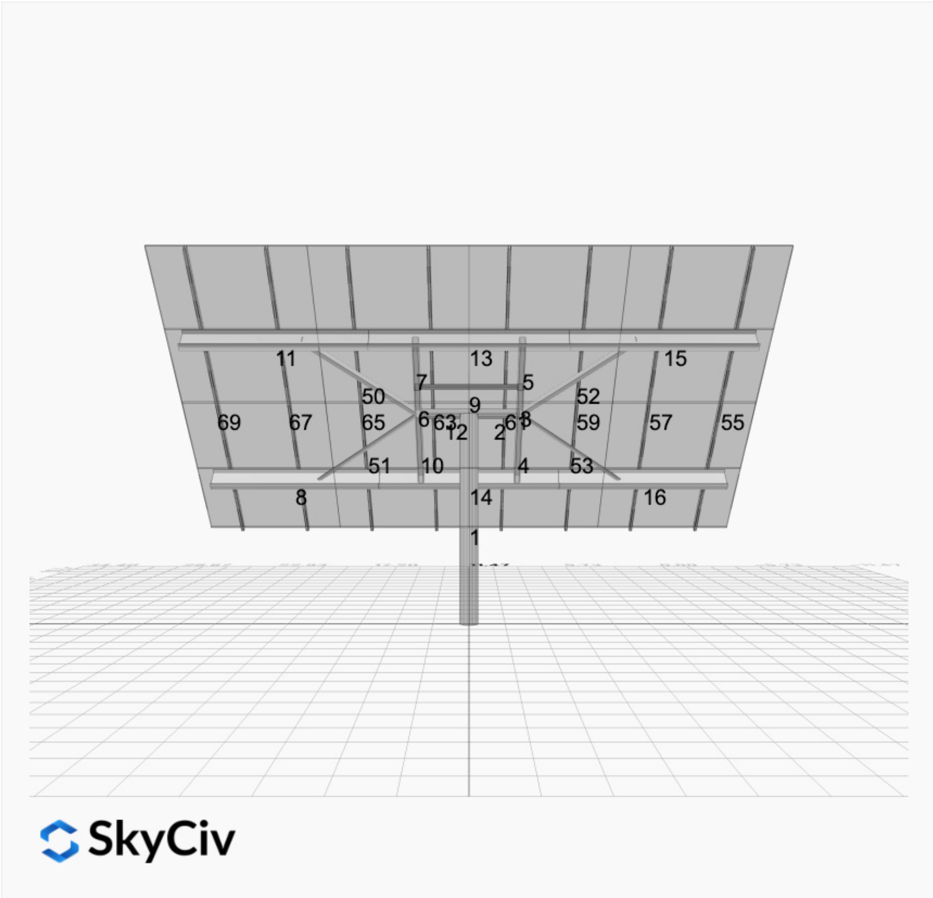


Array Dimensions N/S	15.03 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	45
Front Edge Clearance	3 ft

MT Solar Bill of Materials (1P-0-8TOP-XD-84-L-4Hx4W-STRUTS-GI7K)

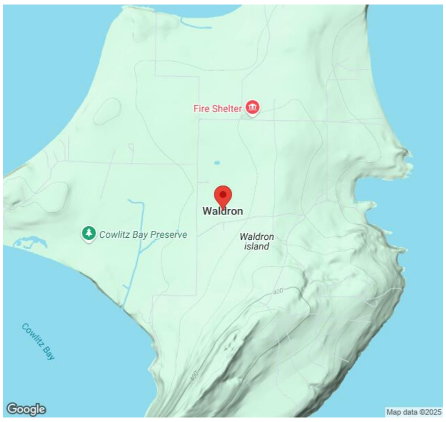
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-XD	H-Frame Assembly-XD	1
MTS-XD-Wing-84	84IN XD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	4

Rail Bill of Materials

Part	Qty
Rails (180in)	8
Rail Attachment	16
Module Mid Clamp	24
Module End Clamp	16

Part	Qty
Ground Lug	4

Site Details:



Site Address: Waldron, WA 98245, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Winter Tilt Angle:	45
Front Edge Clearance:	3
Total Array Height at Tilt:	13.63 ft
Total Frame Length:	21.50 ft
Frame Weight:	1559 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	22.93 ft
Rail Length:	180.40 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	8.32 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: 7.50 ft
Foundation Volume:	4.444 y^3

Site Info

Risk Category:	I
Exposure:	D
Soil Classification:	sand
Site Location:	Waldron, WA 98245, USA
Wind Speed:	95 mph
Snow Load:	25 psf

Design Disclaimer

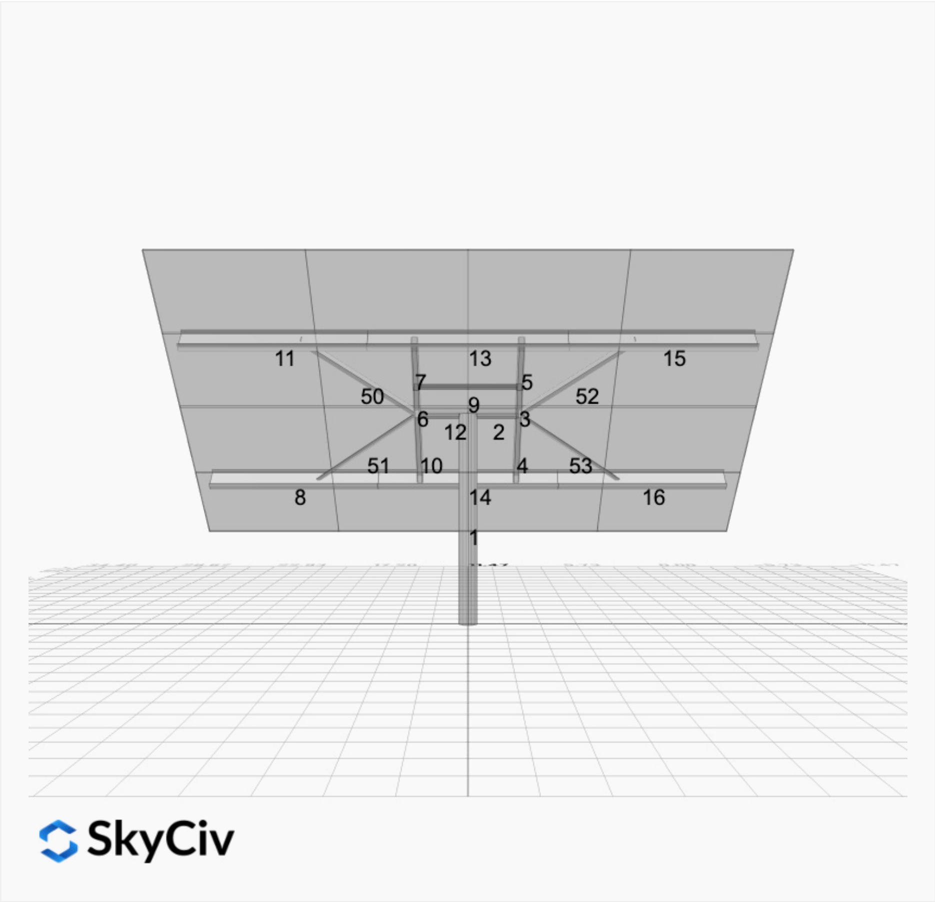
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

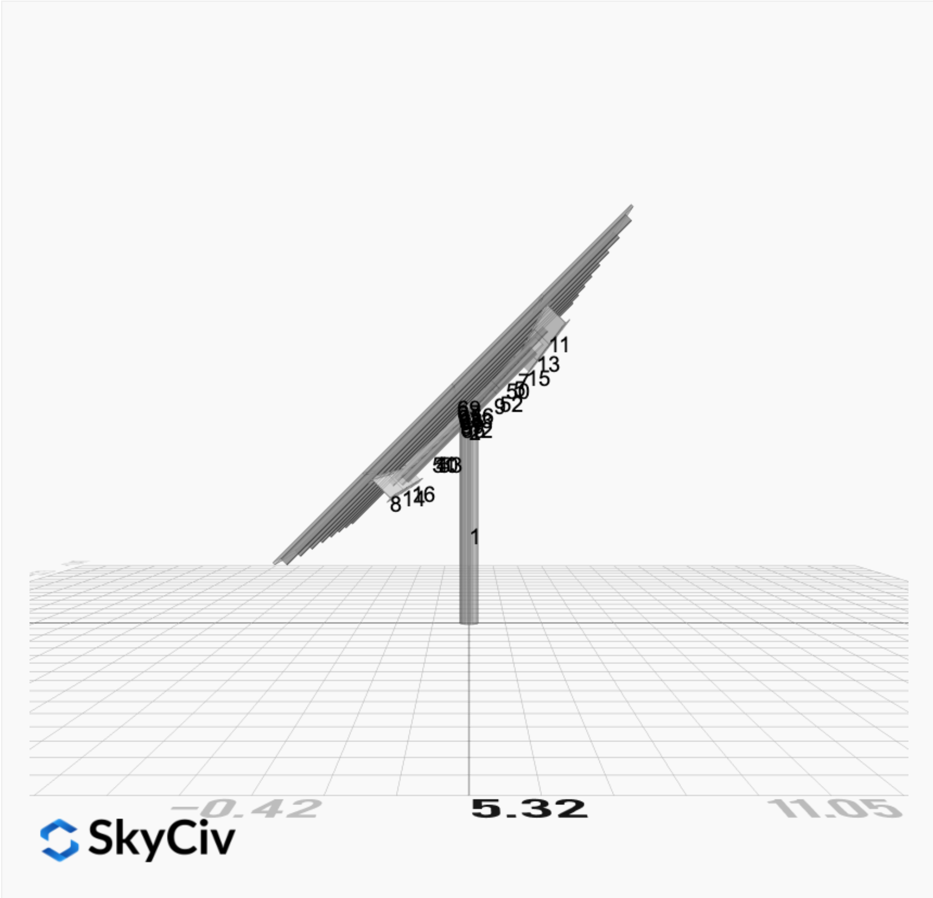
AutoDesigner Input

{"product_type":"Beam","designer_name":"Katee Wood","designer_email":"team@fmsolar.com","designer_phone":"360-422-5610","project_id":"Jonathan Cook - V2Jb","site_address":"Waldron, WA 98245, USA","module_width":44.6,"module_length":67.8,"number_rows":"4","number_columns":4,"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":45,"ground_clearance":3,"risk_category":"I","exposure_category":"D","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false,"wind_speed_override":95,"snow_load_override":25,"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)

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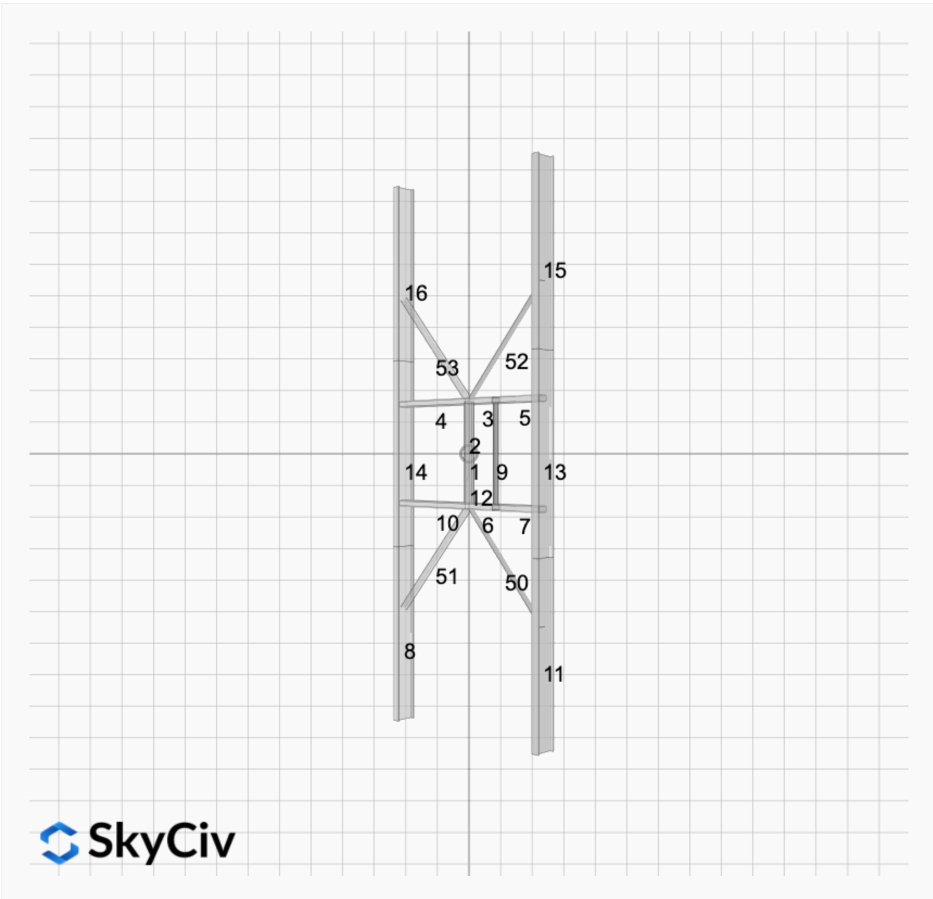
15
1
8
16
-0.42
5.32
11.05
SkyCiv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6
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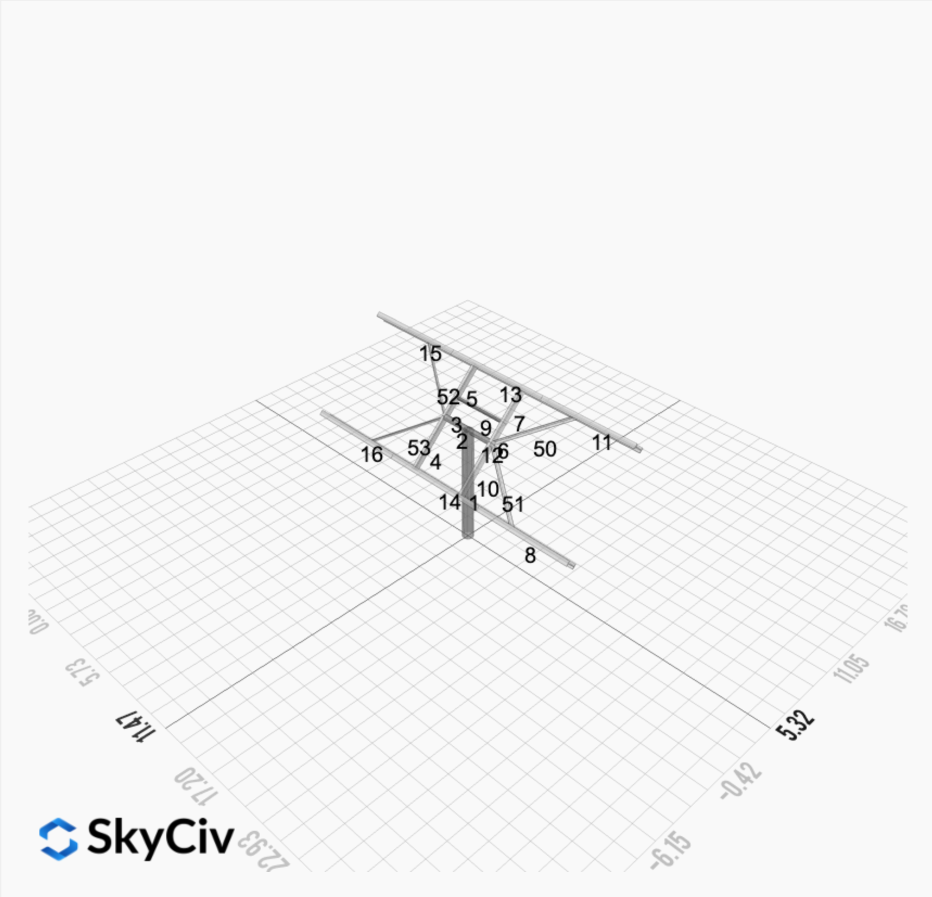
11
SkyCiv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4
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14
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5.32
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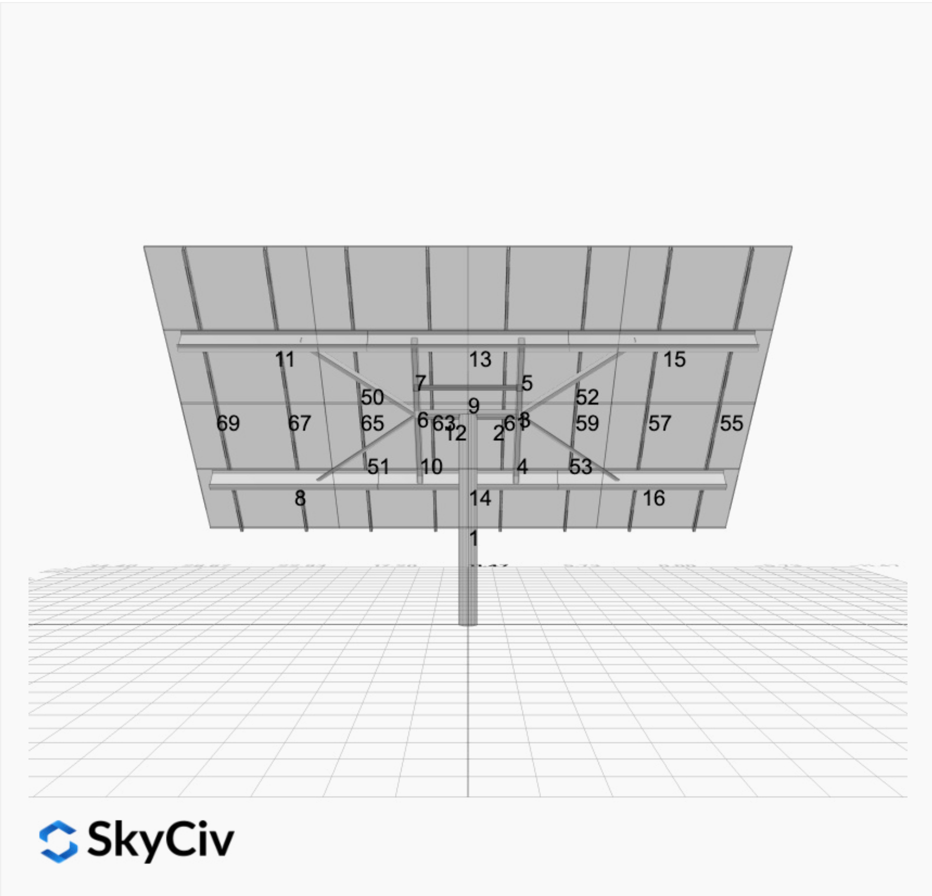
-6.15
SkyCiv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63
12
2
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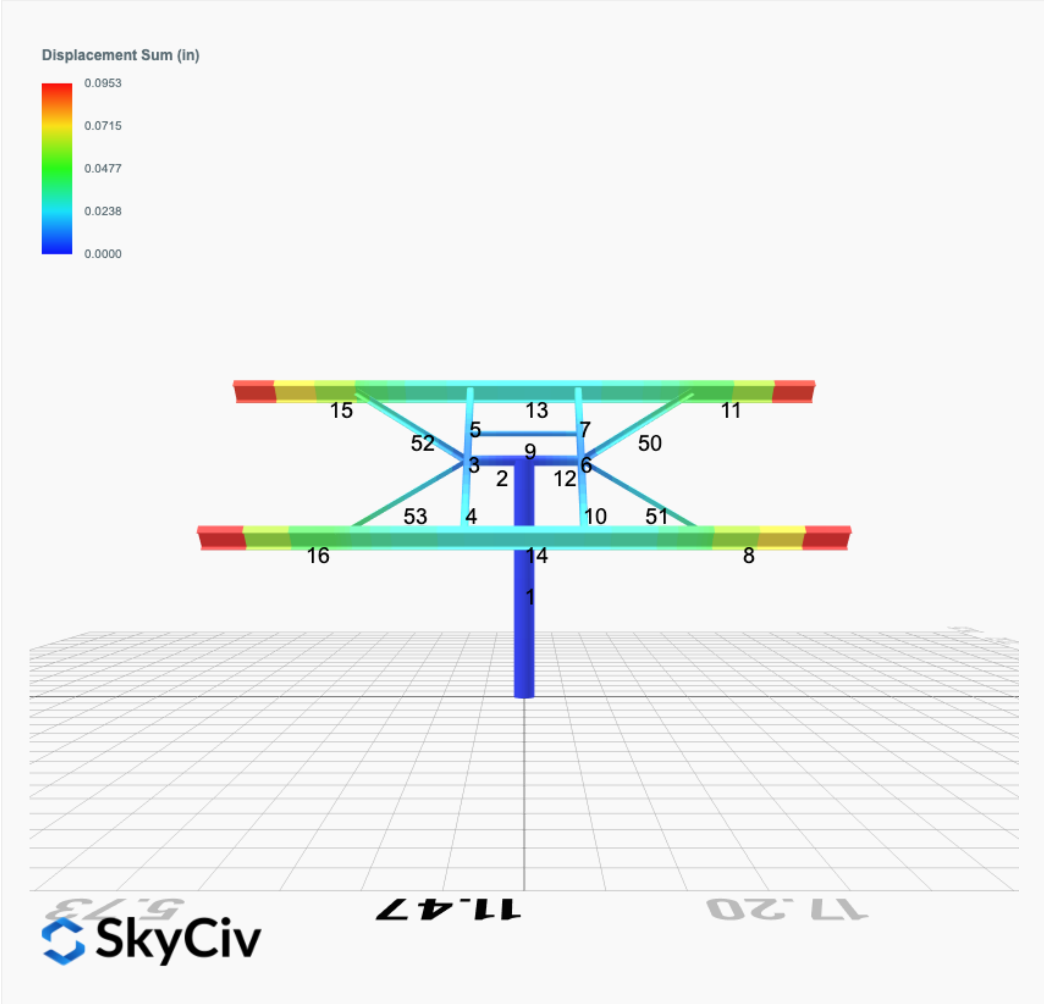
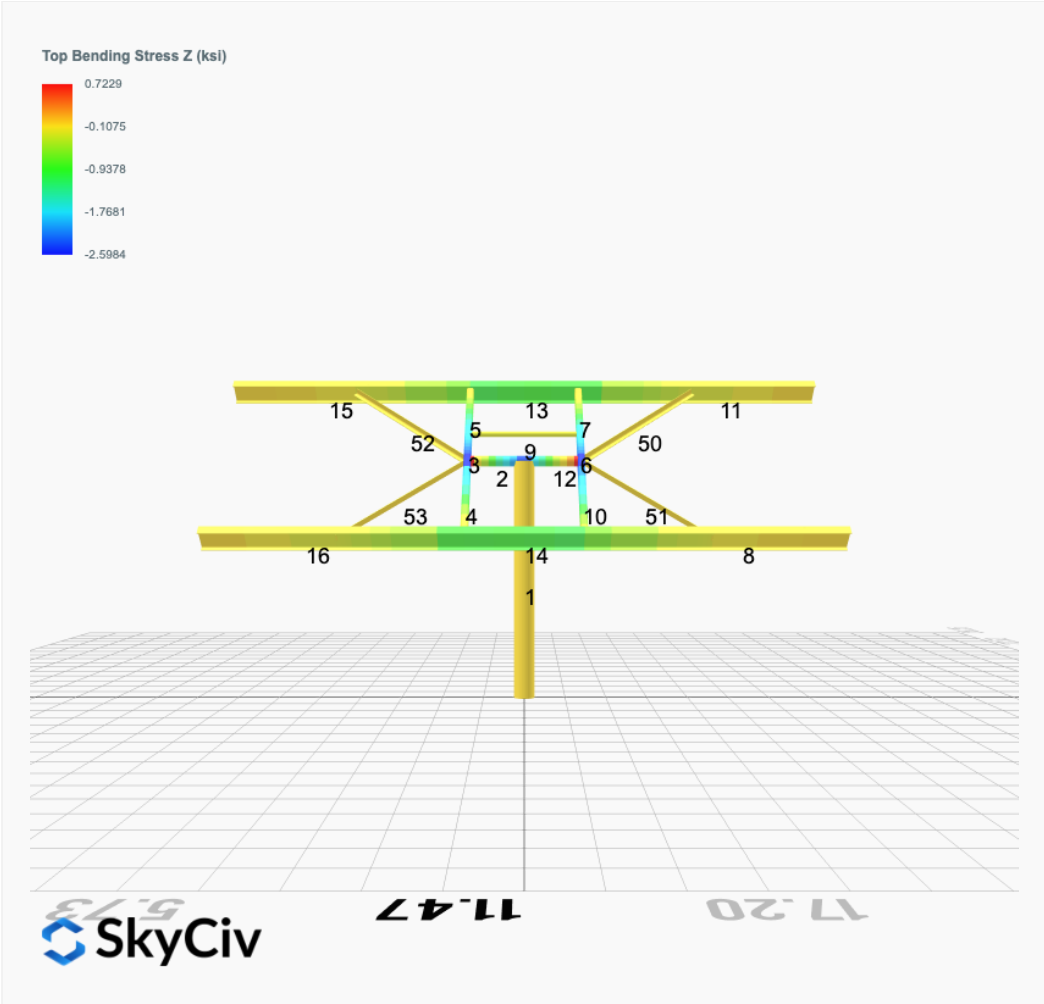
55
51
10
4
53
8
14
16
1
SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
0.7229
-0.1075
-0.9378
-1.7681
-2.5984
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13
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7
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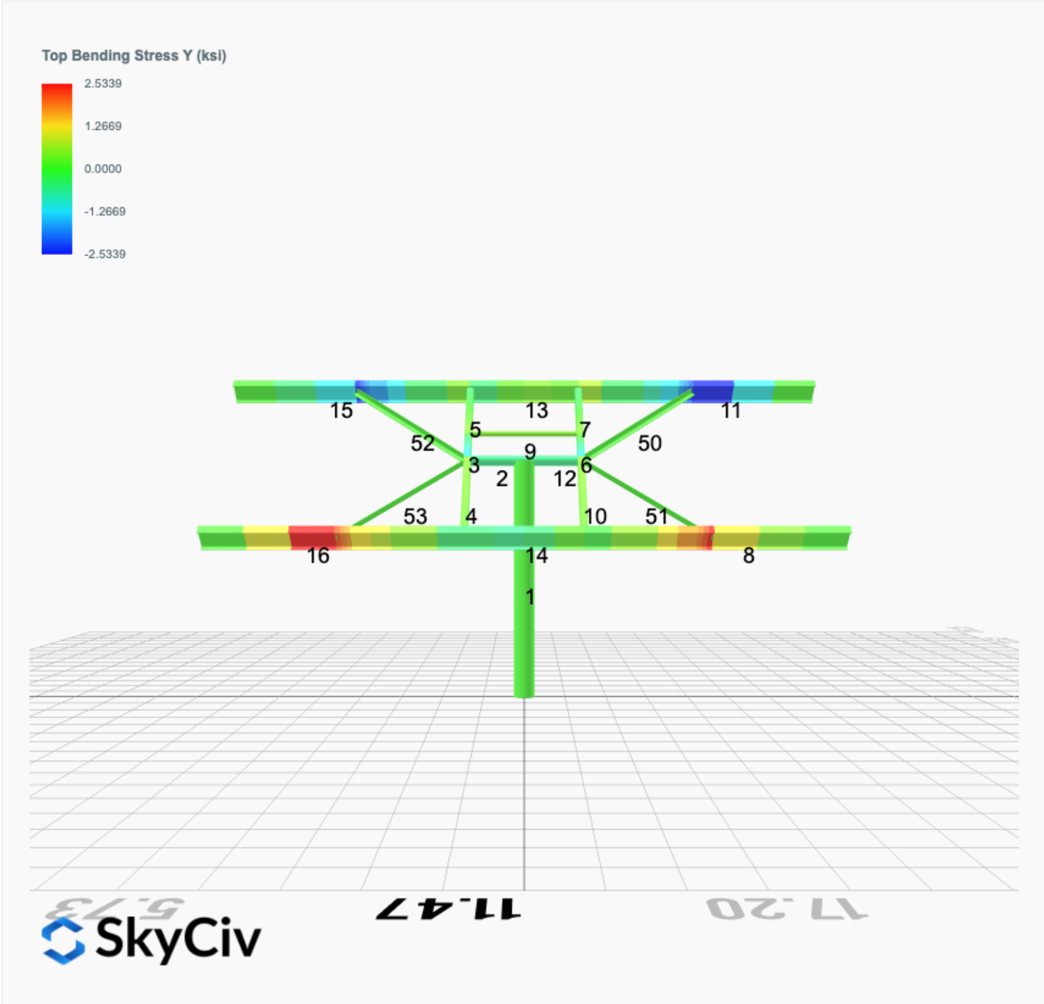
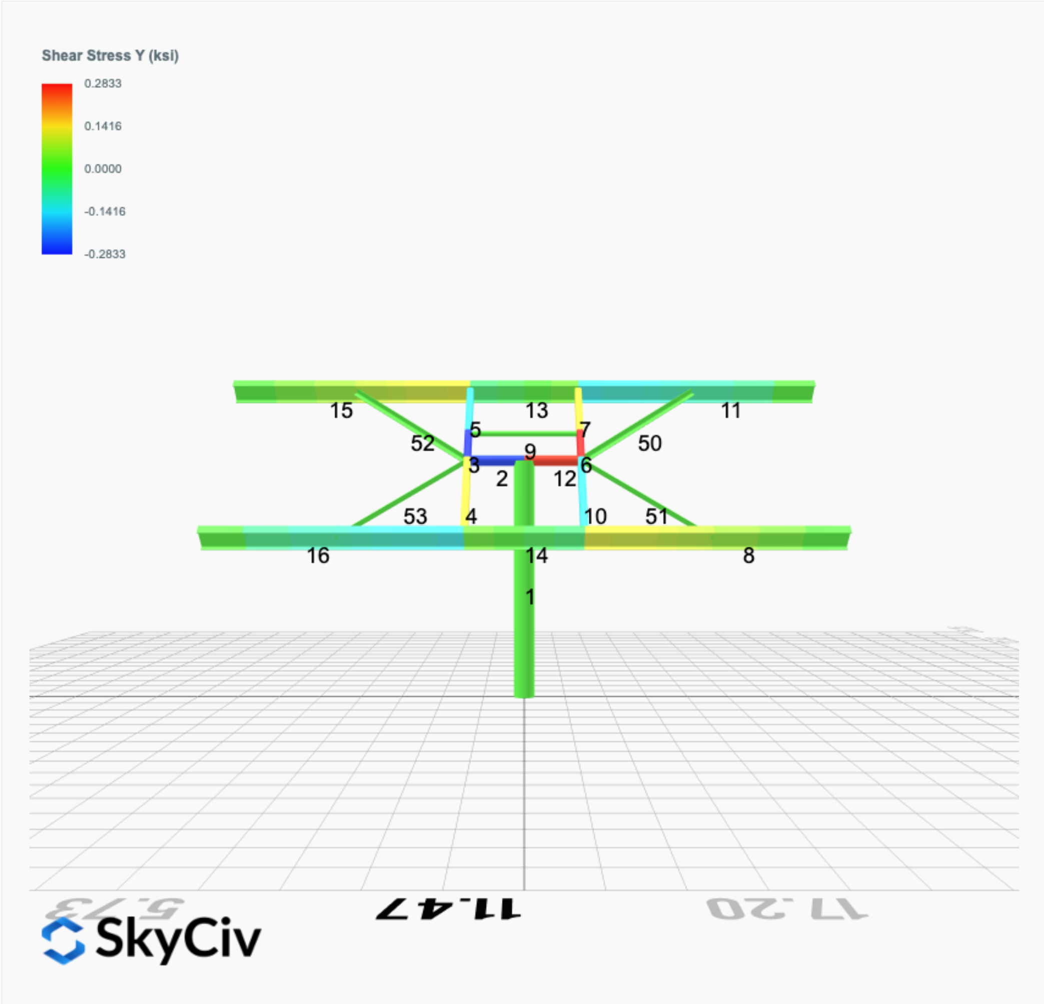
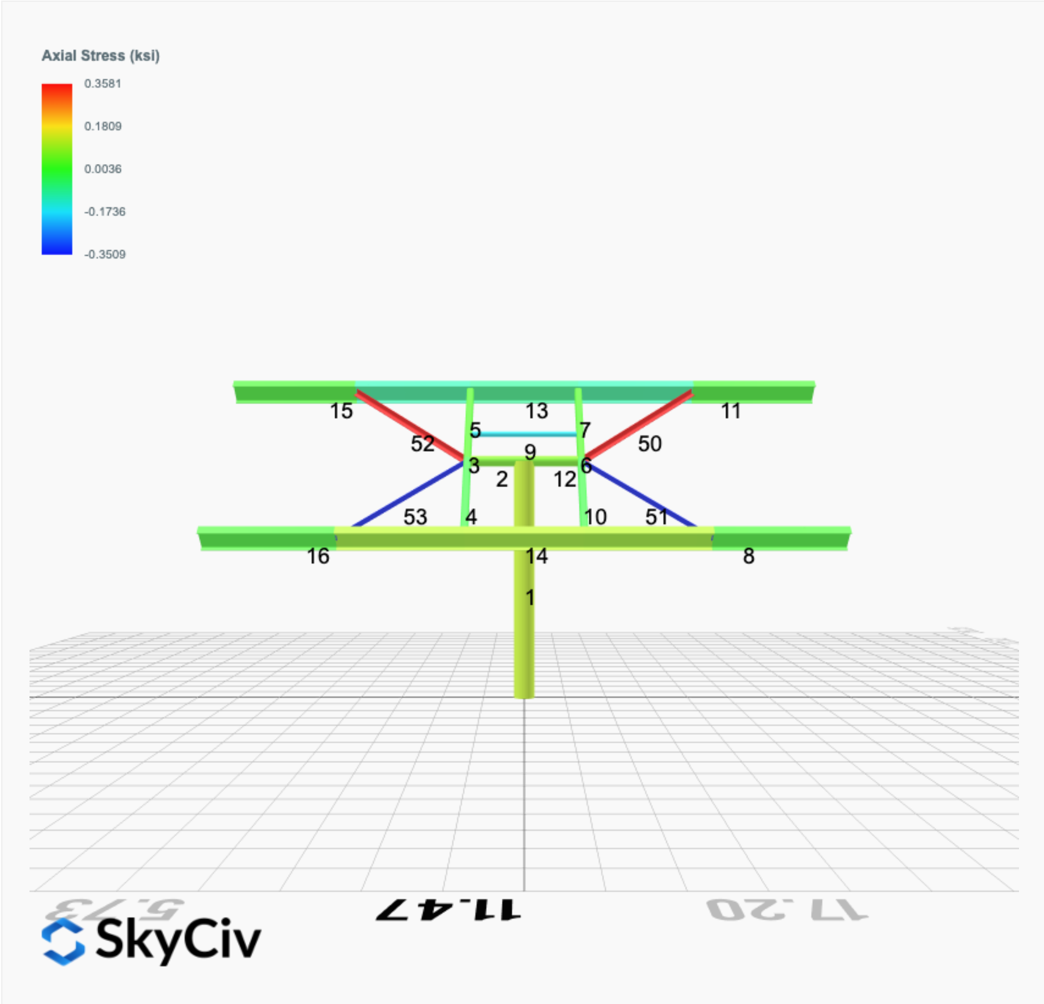
9
3
6
2
12
53
4
10
51
16
14
8
1


Top Bending Stress Y (ksi)
2.5339
1.2669
0.0000
-1.2669
-2.5339
15
13
11
52
5
7
50
3
9
6
2
12
53
4
10
51
16
14
8
1
11.47
SkyCiv


Shear Stress Y (ksi)
0.2833
0.1416
0.0000
-0.1416
-0.2833
15
13
11
52
5
7
50
3
9
6
2
12
53
4
10
51
16
14
8
1
11.47
SkyCiv


Axial Stress (ksi)
0.3581
0.1809
0.0036
-0.1736
-0.3509
15
13
11
5
7
52
50
3
9
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	2.8396	0.0000	-0.0000	-0.0000	0.0275
ULS: 2. D + L	0.0000	2.8396	0.0000	-0.0000	-0.0000	0.0275
ULS: 3. D + (S or Lr or R)	0.0000	4.2358	0.0000	-0.0000	-0.0000	0.0306
ULS: 3. D + (S or Lr or R)	0.0000	2.8396	0.0000	-0.0000	-0.0000	0.0275
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.8867	0.0000	-0.0000	-0.0000	0.0298
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.8396	0.0000	-0.0000	-0.0000	0.0275
ULS: 5b. D + 0.7E	0.0000	2.8396	0.0000	-0.0000	-0.0000	0.0275
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.8867	0.0000	-0.0000	-0.0000	0.0298
ULS: 8. 0.6D + 0.7E	0.0000	1.7037	0.0000	-0.0000	-0.0000	0.0165
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.8733	8.7128	0.0000	-0.0000	-0.0000	51.4094
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.8733	8.7128	0.0000	-0.0000	-0.0000	51.4094
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.2487	-1.4091	0.0000	-0.0000	-0.0000	-33.7218
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7489	-0.9093	0.0000	-0.0000	-0.0000	-41.4373
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.4049	8.2916	0.0000	-0.0000	-0.0000	38.5663
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.4049	8.2916	0.0000	-0.0000	-0.0000	38.5663
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1866	0.7002	0.0000	-0.0000	-0.0000	-25.2822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8117	1.0751	0.0000	-0.0000	-0.0000	-31.0688
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.4049	7.2445	0.0000	-0.0000	-0.0000	38.5640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.4049	7.2445	0.0000	-0.0000	-0.0000	38.5640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1866	-0.3470	0.0000	-0.0000	-0.0000	-25.2845
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8117	0.0279	0.0000	-0.0000	-0.0000	-31.0711
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.8733	7.5769	0.0000	-0.0000	-0.0000	51.3984
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.8733	7.5769	0.0000	-0.0000	-0.0000	51.3984
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.2487	-2.5450	0.0000	-0.0000	-0.0000	-33.7328
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7489	-2.0451	0.0000	-0.0000	-0.0000	-41.4483

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	13.8943
Shear X	-9.7888
Shear Z	0.0000
Moment X	0.0013
Moment Y (Twist)	0.0011
Moment Z	86.1860

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	8.7128
Shear X	-5.8733
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	51.4094

Project Details

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

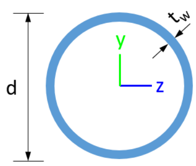
User Name: sales@mtsolar.us
Project Name: Jonathan Cook - V2Jb
Unit System: imperial

Design Input Information

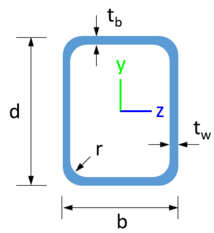
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

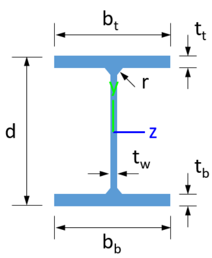
Section Dimensions



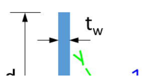
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
10	8in Pipe Sch 80	8.63	0.50					

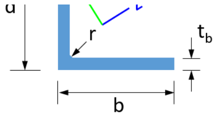


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



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
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
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	10	17.46	17.46	8.32	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.49,1.67,1.67,1.66,1.66	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
8	20	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.30,2.30,2.30,2.31,2.30,2.30,2.29,2.30,2.30,2.30,2.29,2.29,2.30,2.30,2.29,2.26,2.30,2.30,2.30,2.30	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.49,1.67,1.67,1.66,1.66	300	200	1
11	20	7.00	7.00	7.00	2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.29,2.30,2.30,2.30,2.29,2.30,2.30,2.30,2.30,2.30	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.04,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.10,1.02,1.02,1.02,1.01,1.02,1.02,1.02,1.02	300	200	1
15	20	7.00	7.00	7.00	2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.30,2.29,2.30,2.30,2.30,2.29,2.30,2.30,2.30,2.30,2.30	300	200	1
16	20	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.30,2.30,2.30,2.31,2.30,2.30,2.29,2.30,2.30,2.30,2.29,2.29,2.30,2.30,2.29,2.26,2.30,2.30,2.30,2.30	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	574.32	389.79	123.94	123.94	172.30	172.30
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	68.92	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	68.92	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	31.12	6.46	56.26	44.91
14	159.30	97.43	30.92	6.46	56.26	44.91
15	159.30	68.92	46.90	6.46	56.26	44.91
16	159.30	68.92	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	0.036	0.695	0.000	0.057	0.000	0.713	#13	0.364	Not Required	Pass
2	0.003	0.430	0.343	0.089	0.065	0.773	#13	0.054	Not Required	Pass
3	0.001	0.801	0.069	0.080	0.029	0.850	#13	0.046	Not Required	Pass
4	0.001	0.715	0.021	0.071	0.002	0.731	#13	0.082	Not Required	Pass
5	0.001	0.497	0.017	0.080	0.007	0.508	#13	0.076	Not Required	Pass
6	0.001	0.801	0.069	0.080	0.029	0.850	#13	0.046	Not Required	Pass
7	0.001	0.497	0.017	0.080	0.007	0.509	#13	0.076	Not Required	Pass
8	0.012	0.186	0.124	0.044	0.009	0.202	#13	0.535	Not Required	Pass
9	0.014	0.069	0.069	0.001	0.000	0.142	#13	0.137	Not Required	Pass
10	0.001	0.715	0.021	0.071	0.002	0.731	#13	0.082	Not Required	Pass
11	0.005	0.208	0.124	0.050	0.009	0.217	#13	0.357	Not Required	Pass
12	0.003	0.430	0.343	0.089	0.065	0.773	#13	0.054	Not Required	Pass
13	0.005	0.505	0.024	0.062	0.004	0.516	#13	0.204	Not Required	Pass
14	0.009	0.461	0.020	0.055	0.004	0.467	#13	0.306	Not Required	Pass
15	0.005	0.208	0.124	0.050	0.009	0.217	#13	0.357	Not Required	Pass
16	0.012	0.186	0.124	0.044	0.009	0.202	#13	0.357	Not Required	Pass
50	0.136	0.009	0.004	0.002	0.001	0.147	#21	0.783	Not Required	Pass
51	0.027	0.005	0.014	0.001	0.001	0.043	#21	0.522	Not Required	Pass
52	0.136	0.009	0.004	0.002	0.001	0.147	#24	0.783	Not Required	Pass
53	0.027	0.005	0.014	0.001	0.001	0.043	#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/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

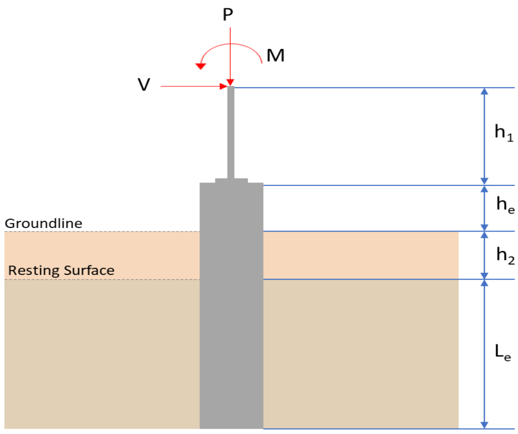
SkyCiv Foundation Design

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
$L = 7.5\ \mathrm{ft}$ - 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)	8.713	13.894
V_x (kip)	-5.873	-9.789
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.001
M_z (kipft)	51.409	86.186

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,

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

$$H_o = \frac{(-5.873\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.93519\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(51.409 \text{ kipft}) + ((-5.873 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.1861 \text{ kipft/ft}$$

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:
$L_{e,x} = 6.5842 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(6.5842 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.584 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.584 \text{ ft})}{(7.5 \text{ ft})}$$

$$Ratio = 0.87787$$

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

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

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

$$q = \frac{(8.713 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.54456 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.54456 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.27228$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.93519\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 8.1861\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.1861\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.93519\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{(6 \times (8.1861\ \mathrm{kipft/ft})) + (4 \times (-0.93519\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))}$$

$$a = 5.2272\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.1861\ \mathrm{kipft/ft})) + (3 \times (-0.93519\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]^2}{(7.5\ \mathrm{ft})^2 \times [(3 \times (8.1861\ \mathrm{kipft/ft})) + (2 \times (-0.93519\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}$$

$$p = 0.17341\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (8.1861\ \mathrm{kipft/ft})) + ((-0.93519\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}{(7.5\ \mathrm{ft})^2}$$

$$s = 0.99823\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.2272\ \mathrm{ft})}{2}$$

$$p_a = 0.39204\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.17341\ \mathrm{kip/ft^2}\right)}{\left(0.39204\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.44232$$

Status: **PASS**
Ratio: **0.440**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.5\ \mathrm{ft})$$

$$p_s = 1.125\ \mathrm{kip/ft^2}$$

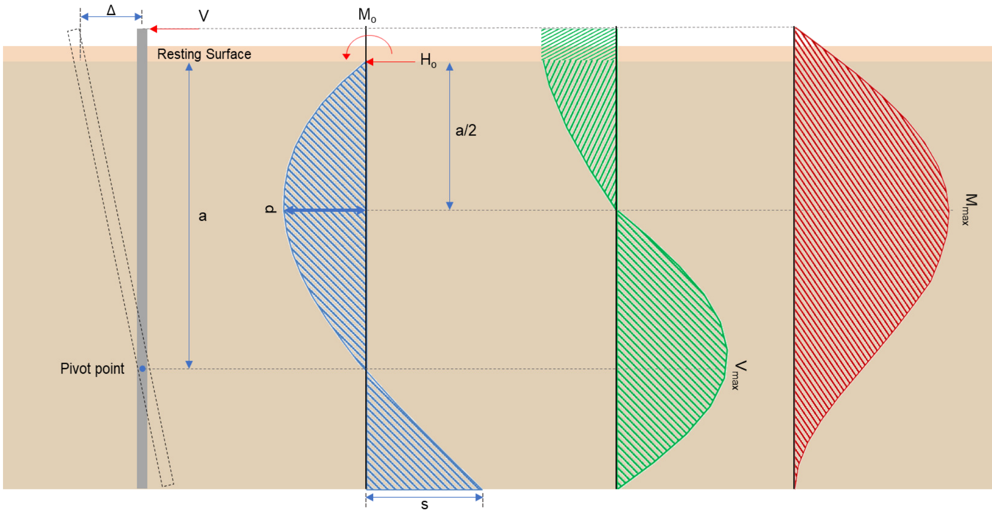
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.99823\ \mathrm{kip/ft^2}\right)}{\left(1.125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88731$$

Status: **PASS**
Ratio: **0.890**



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

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.789\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -1.5588\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(86.186\ \text{kipft}) + ((-9.789\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 13.724\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(13.724\ \text{kipft/ft})}{(-1.5588\ \text{kip/ft})}$$

$$E = 8.8044\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.724\ \text{kipft/ft}) \times (7.5\ \text{ft})) + \left(3 \times (-1.5588\ \text{kip/ft}) \times (7.5\ \text{ft})^2\right)}{(6 \times (13.724\ \text{kipft/ft})) + (4 \times (-1.5588\ \text{kip/ft}) \times (7.5\ \text{ft}))}$$

$$a = 5.2264\ \text{ft}$$

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

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

$$V_{max} = ((-1.5588\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8044\ \text{ft})}{(7.5\ \text{ft})} + 3\right) \times \left(\frac{(5.2264\ \text{ft})}{(7.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (8.8044\ \text{ft})}{(7.5\ \text{ft})} + 2\right) \times \left(\frac{(5.2264\ \text{ft})}{(7.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 17.065\ \text{kip}$$

$$V_{max} = 17.065 \text{ kip}$$

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

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

$$M_{max} = ((-1.5588 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(8.8044 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.2264 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8044 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.2264 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8044 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.2264 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 59.772 \text{ kipft}$$

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,

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

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

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

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (7.5 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7.5 \text{ ft})^2\right)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(7.5 \text{ ft})} \right) \times \left(\frac{(5 \text{ ft})}{(7.5 \text{ ft})} - 1 \right) \times \left(\frac{(5 \text{ ft})}{(7.5 \text{ ft})} \right)^2$$

$$V_{max} = 0.00015098 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(5 \text{ ft})}{2 \times (7.5 \text{ ft})}^3 \right) + \left(3 \times \frac{(5 \text{ ft})}{2 \times (7.5 \text{ ft})}^4 \right) \right]$$

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

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5$ ksi - Concrete strength,
$f_{yk} = 60$ 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,
$A_g = 2304$ 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.894\text{ kip})}{(0.65)\times(0.8)} - \left(0.85\times(2.5\text{ ksi})\times\left(2304\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85\times(2.5\text{ ksi}))}, \left(0.08\times\left(2304\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.134\text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.134\text{ in}^2\right), \left(0.0018\times\left(2304\text{ in}^2\right)\right)\right]$$

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

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

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

$$Ratio = \frac{(13.894 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0051937$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48$ 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.894 \text{ kip} \rightarrow 13894 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13894 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.34 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.34 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.34 \text{ kip}$$

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)

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

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

	$$V_{s,a} = 737.28\ \mathrm{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$		
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$ $$V_{s,b} = 50.894\ \mathrm{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$		
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.34\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 111.3\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 17.065\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(17.065\ \mathrm{kip})}{(111.3\ \mathrm{kip})}$$ $$Ratio = 0.15333$$	Status: **PASS** Ratio: **0.150**	
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$		
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f'_{ck}\ S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \mathrm{kipft}$$		

	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:** $M_{max} = 59.772\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(59.772\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.23947$$	Status: **PASS** Ratio: **0.240**
	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**