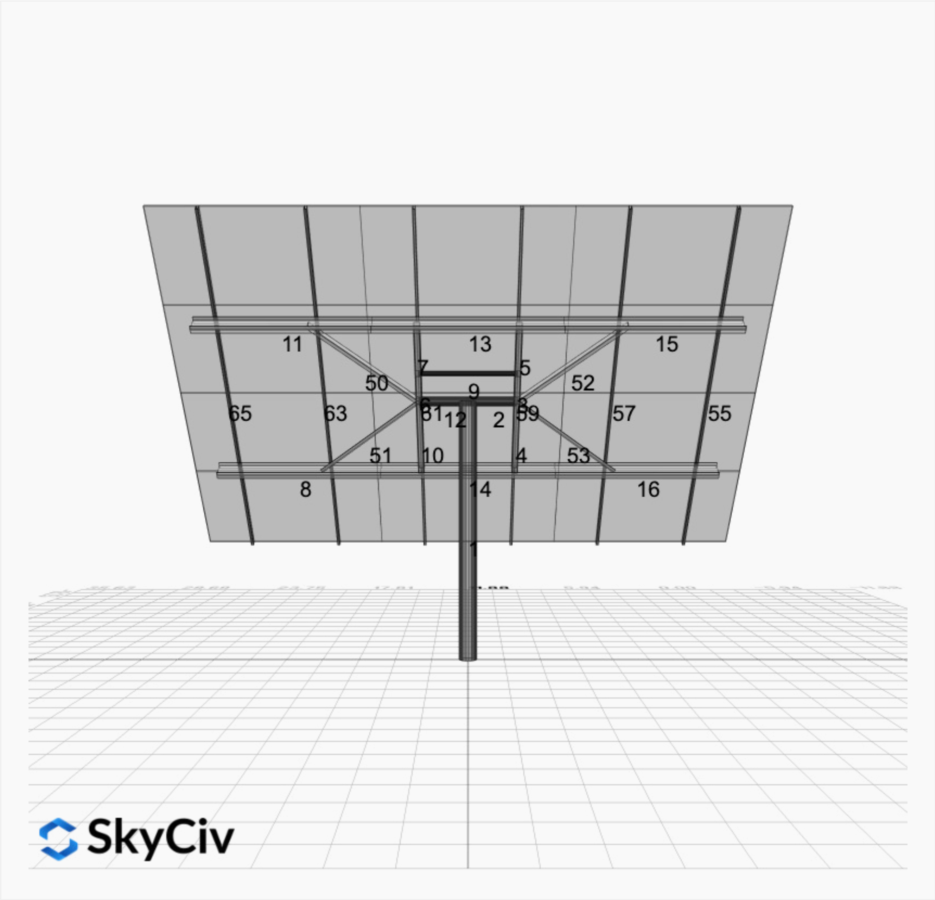
Project Details

Project Name: South Pack - Riley - 4x3 - 710w - NF - Rails - V1Jb
Location: 37 Arundel Rd, Peterborough, NH 03458, USA
Unique ID: 1P-0-8TOP-HD-84-L-4Hx3W-STRUTS-56L0
Dealer: ______________________________

Date: Fri Jan 24 2025
Number of Modules: 12
Number of Poles: 1
Date Sold:
____



Array Dimensions N/S	17.17 ft
Array Dimensions E/W	23.75 ft
Winter Tilt Angle	50
Front Edge Clearance	4 ft

MT Solar Bill of Materials (1P-0-8TOP-HD-84-L-4Hx3W-STRUTS-56L0)

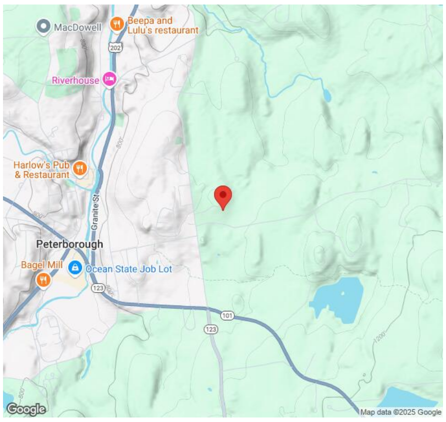
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-84	84IN HD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (204in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12

Part	Qty
Ground Lug	3

Site Details:



Site Address: 37 Arundel Rd, Peterborough, NH 03458, USA

Array Specification

Duty Classification:	HD
Module Width:	51.00 in
Module Length:	94.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	50
Front Edge Clearance:	4
Total Array Height at Tilt:	17.15 ft
Total Frame Length:	21.50 ft
Frame Weight:	1308 lbs
Array Dimensions N/S:	17.17 ft
Array Dimensions E/W:	23.75 ft
Rail Length:	206.00 in
Rail Spacing:	3.96 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	10.58 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: 7.25 ft
Foundation Volume:	4.296 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	37 Arundel Rd, Peterborough, NH 03458, USA
Wind Speed:	104 mph
Snow Load:	84 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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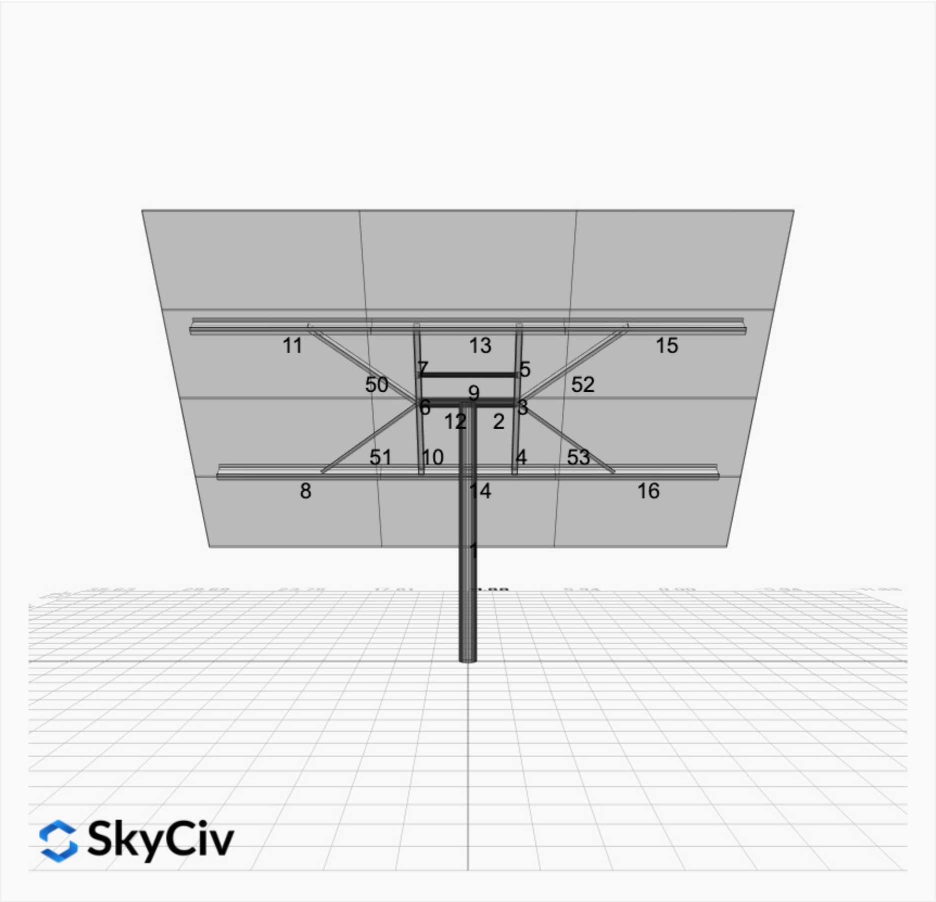
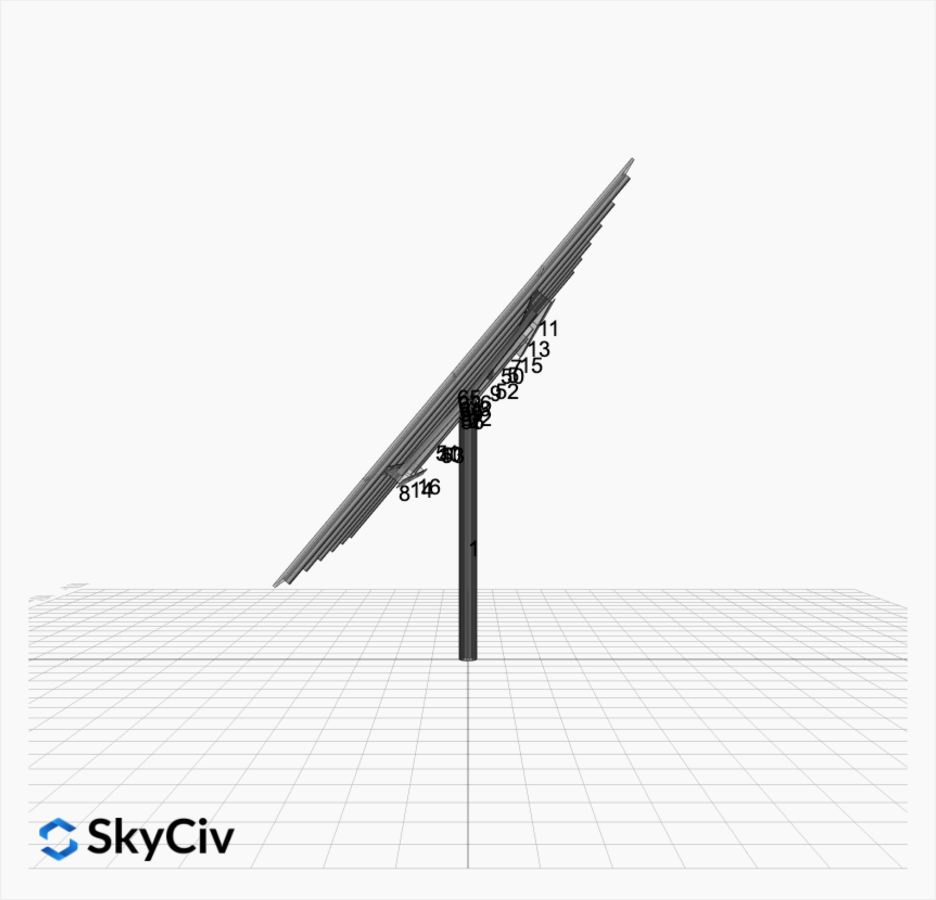
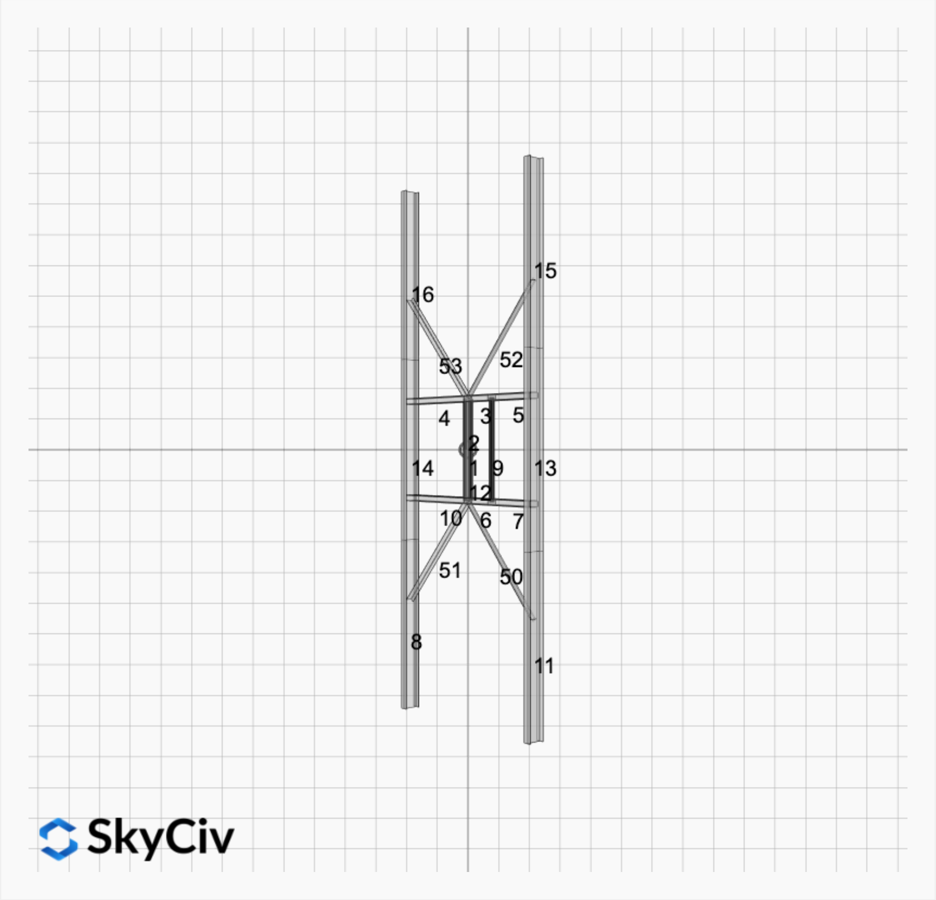
Design Notes:

- 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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50
8
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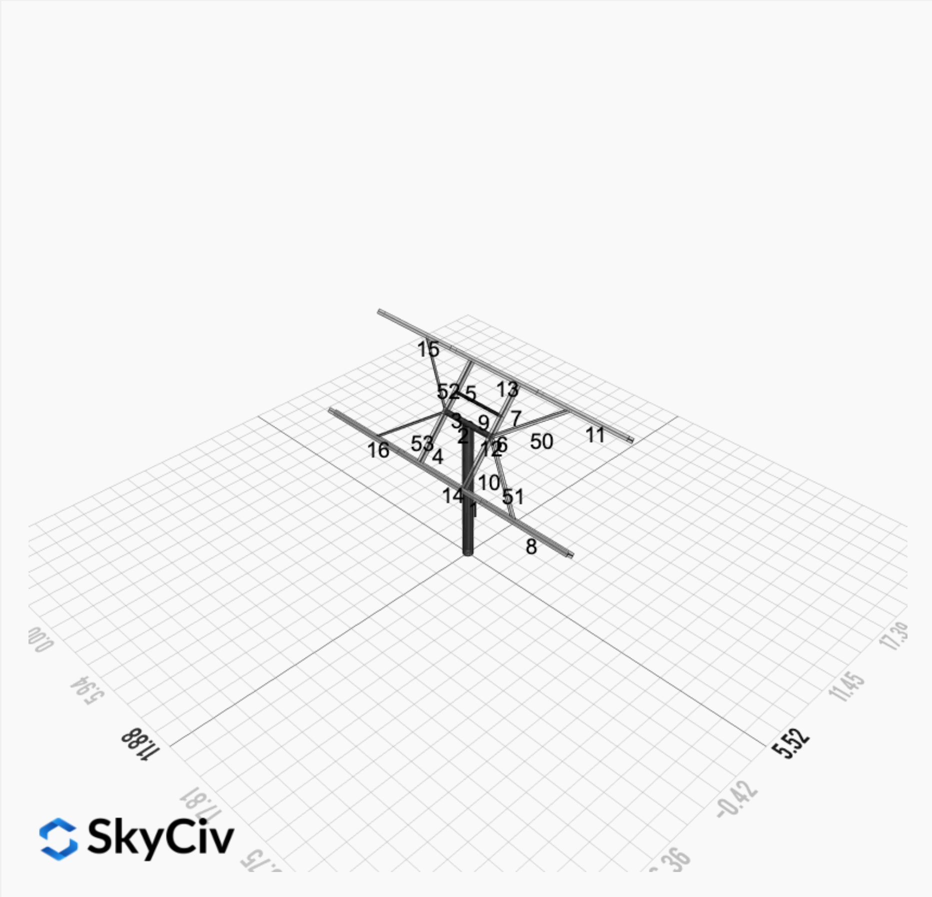
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12
6
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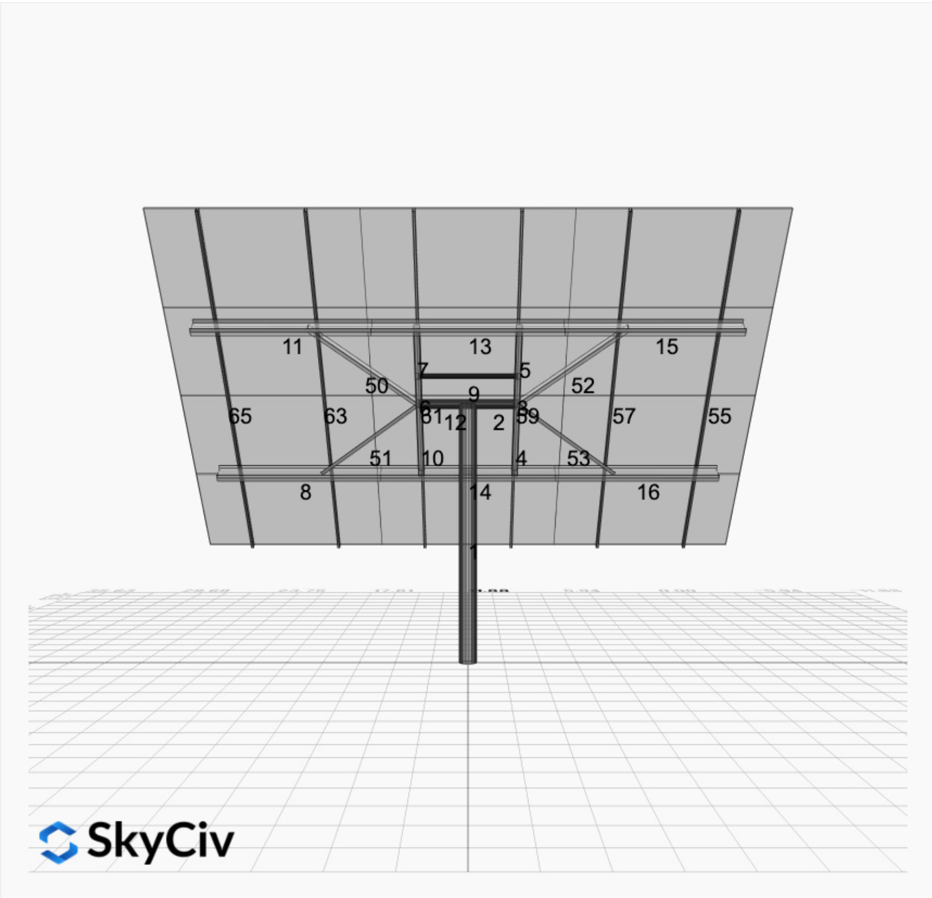
11.88
17.81
5.52
-0.42
11.45
17.39
SkyCiv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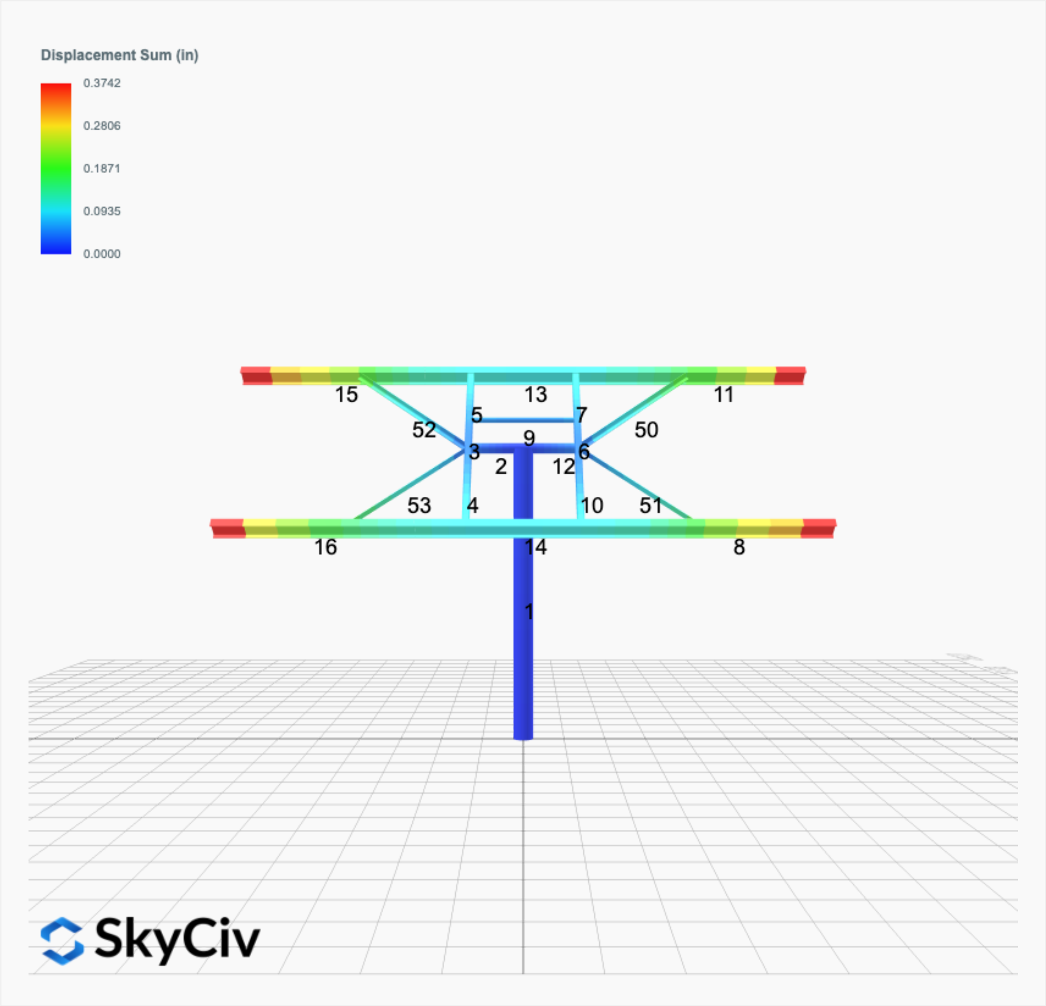
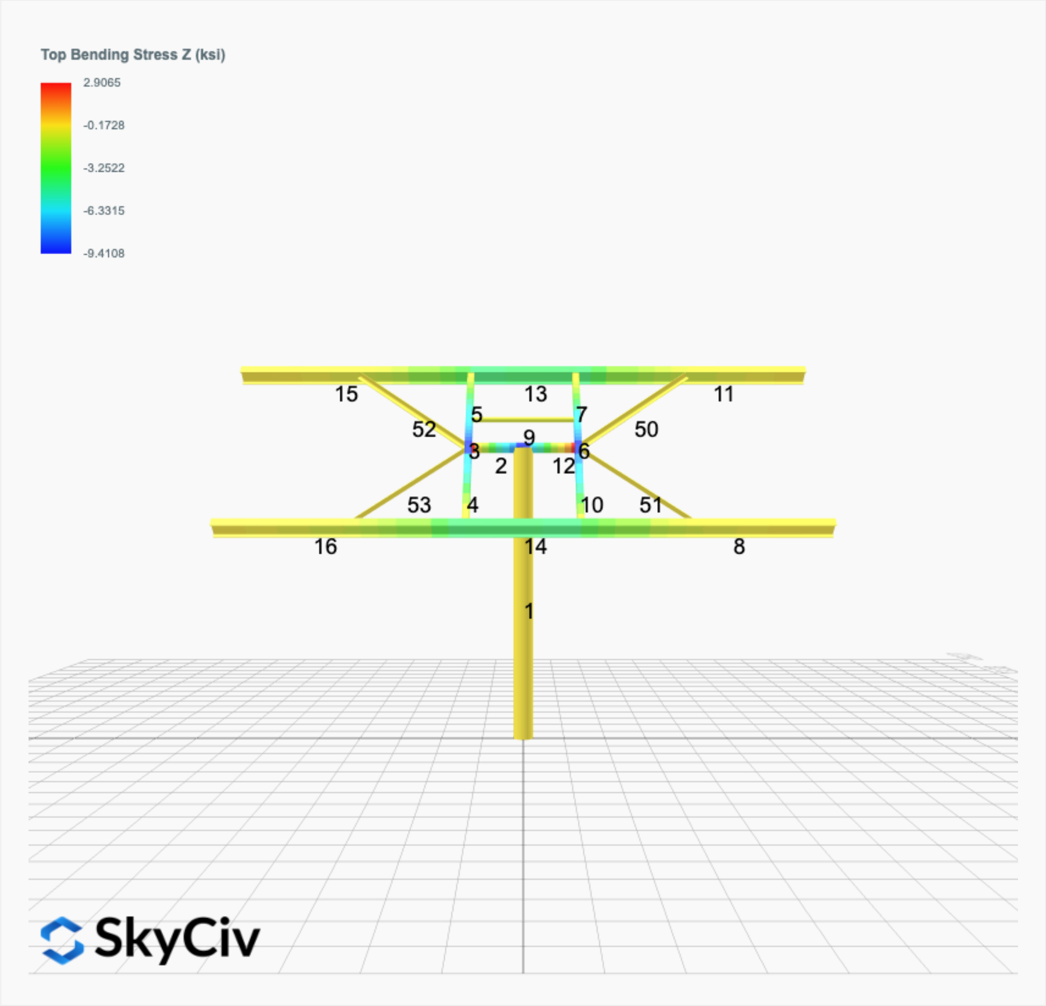
1
SkyCiv

FEM Results (Envelope Worst Case for each member)

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-0.1728
-3.2522
-6.3315
-9.4108
15
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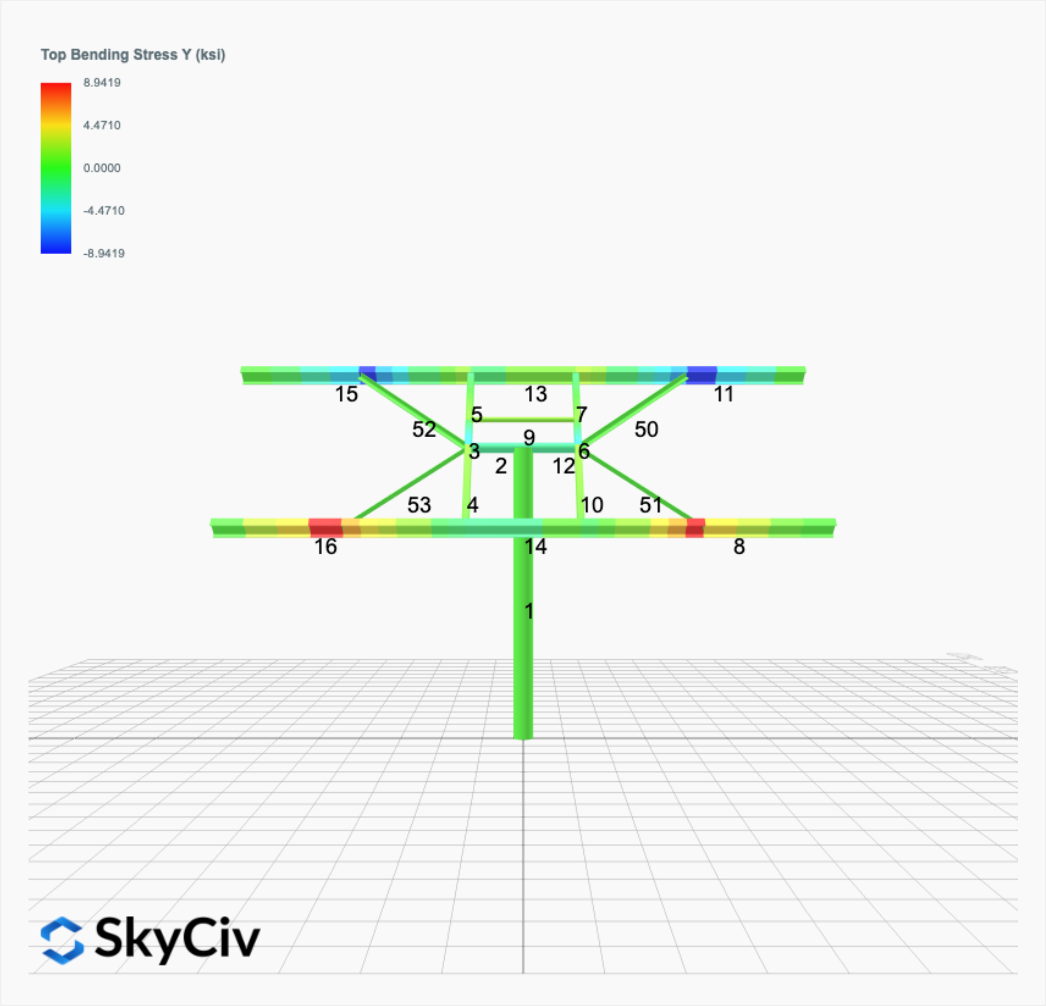
50
9
3
6
2
12
53
4
10
51
16
14
8
1


Top Bending Stress Y (ksi)
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4.4710
0.0000
-4.4710
-8.9419
15
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9
50
3
6
2
12
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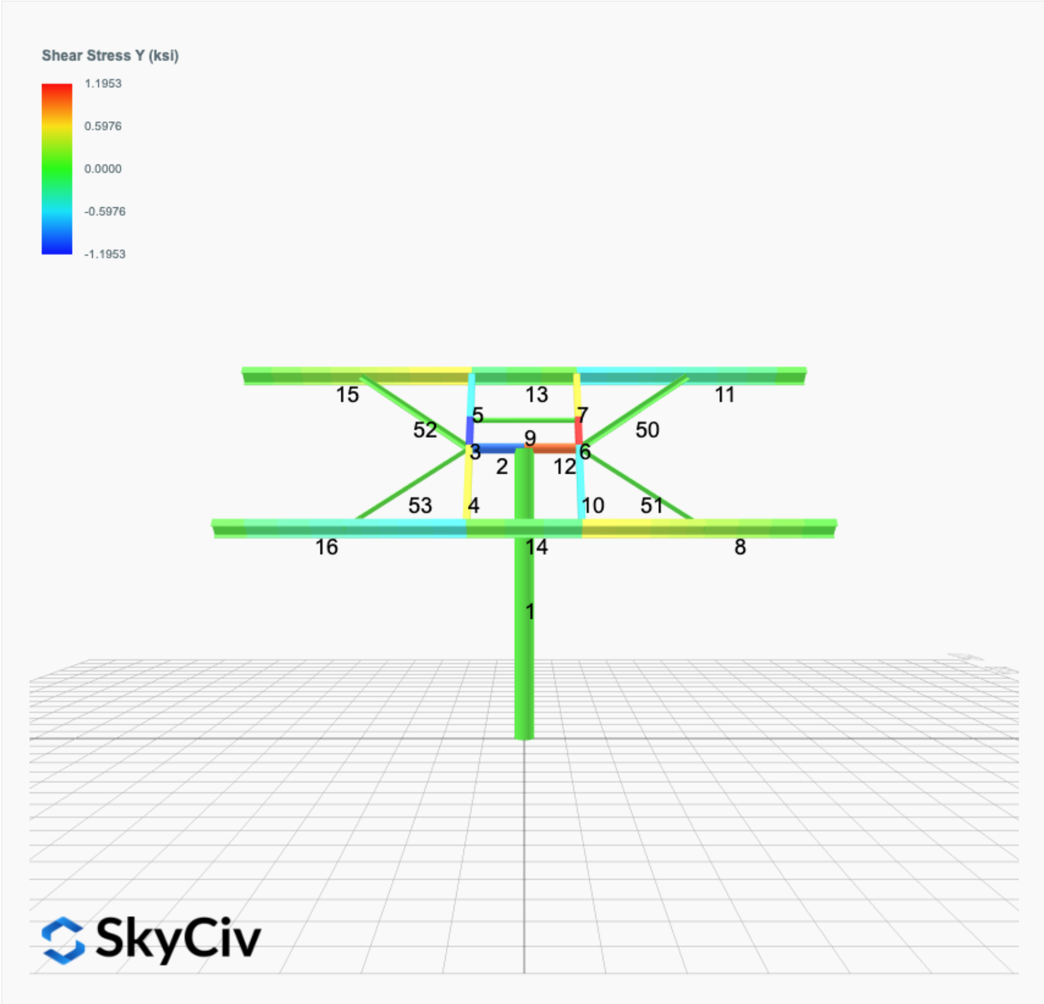
4
10
51
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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0.5976
0.0000
-0.5976
-1.1953
15
13
11
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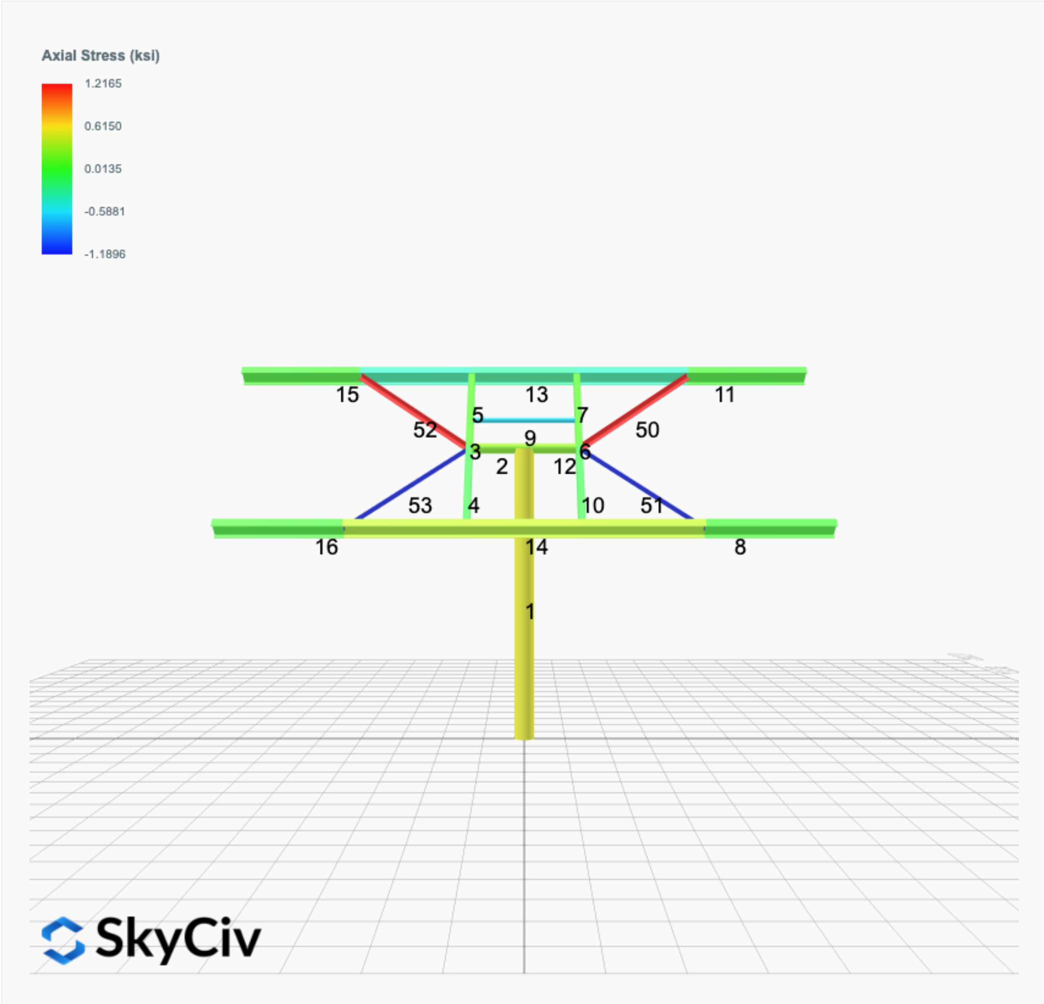
3
6
2
12
53
4
10
51
16
14
8
1
SkyCiv


Axial Stress (ksi)
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0.6150
0.0135
-0.5881
-1.1896
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11
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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	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 2. D + L	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 3. D + (S or Lr or R)	0.0000	7.2115	0.0000	-0.0000	-0.0000	0.0686
ULS: 3. D + (S or Lr or R)	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.1158	0.0000	-0.0000	-0.0000	0.0580
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 5b. D + 0.7E	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.1158	0.0000	-0.0000	-0.0000	0.0580
ULS: 8. 0.6D + 0.7E	0.0000	1.6972	0.0000	-0.0000	-0.0000	0.0157
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.8058	6.0222	0.0000	-0.0000	-0.0000	40.8295
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.8058	-0.3647	0.0000	-0.0000	-0.0000	-39.6829
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8544	8.5109	0.0000	-0.0000	-0.0000	30.6605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	6.1158	0.0000	-0.0000	-0.0000	0.0580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8544	3.7207	0.0000	-0.0000	-0.0000	-29.7238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	6.1158	0.0000	-0.0000	-0.0000	0.0580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8544	5.2238	0.0000	-0.0000	-0.0000	30.6286
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8544	0.4336	0.0000	-0.0000	-0.0000	-29.7557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.8287	0.0000	-0.0000	-0.0000	0.0261
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.8058	4.8907	0.0000	-0.0000	-0.0000	40.8190
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.6972	0.0000	-0.0000	-0.0000	0.0157
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.8058	-1.4962	0.0000	-0.0000	-0.0000	-39.6934
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.6972	0.0000	-0.0000	-0.0000	0.0157

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	13.0681
Shear X	-6.3430
Shear Z	0.0000
Moment X	-0.0016
Moment Y (Twist)	0.0008
Moment Z	69.2386

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	8.5109
Shear X	-3.8058
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	40.8295

Project Details

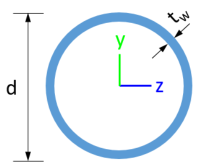
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	South Pack - Riley - 4x3 - 710w - NF - Rails - V1Jb
Unit System:	imperial

Design Input Information

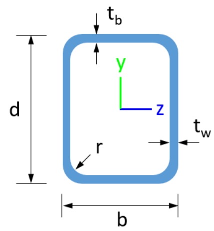
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

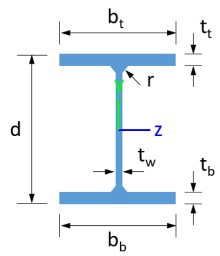
Section Dimensions



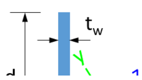
ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					

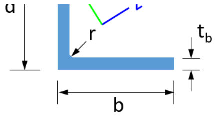


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
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	9	22.21	22.21	10.58	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
4	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
6	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
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9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
11	19	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.32,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.31,2.33,2.33,2.32,2.29,2.32,2.32,2.33,2.31,2.33,2.32,2.33,2.31,2.33	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	19	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	19	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.32,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.31,2.33,2.33,2.32,2.29,2.32,2.32,2.33,2.31,2.33,2.32,2.33,2.31,2.33	300	200	1
16	19	7.00	7.00	7.00	2.32,2.32,2.32,2.31,2.32,2.33,2.32,2.32,2.32,2.32,2.32,2.32,2.31,2.32,2.32,2.31,2.28,2.31,2.32,2.32,2.31,2.32,2.32,2.33,2.31,2.33	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 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	207.08	83.29	83.29	113.39	113.39
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	64.15	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	64.15	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	23.29	6.12	40.24	43.62
14	133.20	85.85	23.29	6.12	40.24	43.62
15	133.20	64.15	32.87	6.12	40.24	43.62
16	133.20	64.15	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	0.063	0.831	0.000	0.056	0.000	0.858	#13	0.454	Not Required	Pass
2	0.009	0.461	0.299	0.107	0.053	0.706	#13	0.053	Not Required	Pass
3	0.002	0.692	0.185	0.070	0.077	0.819	#21	0.045	Not Required	Pass
4	0.003	0.685	0.056	0.069	0.005	0.716	#13	0.080	Not Required	Pass
5	0.002	0.430	0.045	0.069	0.018	0.452	#13	0.074	Not Required	Pass
6	0.002	0.692	0.185	0.070	0.077	0.819	#21	0.045	Not Required	Pass
7	0.002	0.430	0.045	0.069	0.018	0.452	#13	0.074	Not Required	Pass
8	0.028	0.200	0.286	0.047	0.019	0.375	#21	0.500	Not Required	Pass
9	0.033	0.059	0.072	0.001	0.000	0.139	#13	0.136	Not Required	Pass
10	0.003	0.685	0.056	0.069	0.005	0.716	#13	0.080	Not Required	Pass
11	0.014	0.201	0.286	0.047	0.020	0.367	#21	0.333	Not Required	Pass
12	0.009	0.461	0.299	0.107	0.053	0.706	#13	0.053	Not Required	Pass
13	0.014	0.451	0.053	0.059	0.010	0.473	#13	0.190	Not Required	Pass
14	0.022	0.454	0.042	0.058	0.009	0.464	#13	0.286	Not Required	Pass
15	0.014	0.201	0.286	0.047	0.020	0.367	#21	0.333	Not Required	Pass
16	0.028	0.200	0.286	0.047	0.019	0.375	#21	0.333	Not Required	Pass
50	0.294	0.009	0.005	0.003	0.002	0.307	#21	0.783	Not Required	Pass
51	0.059	0.004	0.014	0.002	0.002	0.072	#21	0.522	Not Required	Pass
52	0.294	0.009	0.005	0.003	0.002	0.307	#23	0.783	Not Required	Pass
53	0.059	0.004	0.014	0.002	0.002	0.072	#23	0.522	Not Required	Pass

Definitions

Φ Safety factor for tensile

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

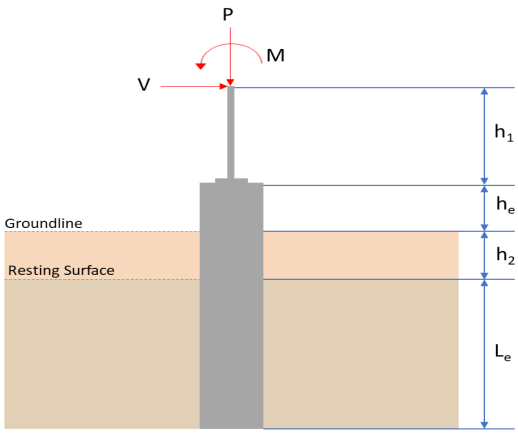
SkyCiv Foundation Design

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
$L = 7.25 \text{ ft}$ - 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)	8.511	13.068
V_x (kip)	-3.806	-6.343
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.002
M_z (kipft)	40.829	69.239

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,

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

$$H_o = \frac{(-3.806 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.829\ \text{kipft}) + ((-3.806\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 6.5014\ \text{kipft/ft}$$

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:
$L_{e,x} = 6.5556\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(6.5556\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 6.556\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 7.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.556\ \text{ft})}{(7.25\ \text{ft})}$$

$$Ratio = 0.90428$$

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

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

$$q = \frac{(8.511\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.53194\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.53194\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.26597$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.8125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.60605\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.5014\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5014\ \mathrm{kipft/ft}) \times (7.25\ \mathrm{ft})) + \left(3 \times (-0.60605\ \mathrm{kip/ft}) \times (7.25\ \mathrm{ft})^2\right)}{(6 \times (6.5014\ \mathrm{kipft/ft})) + (4 \times (-0.60605\ \mathrm{kip/ft}) \times (7.25\ \mathrm{ft}))}$$

$$a = 5.021\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times \left[(4 \times (6.5014\ \mathrm{kipft/ft})) + (3 \times (-0.60605\ \mathrm{kip/ft}) \times (7.25\ \mathrm{ft}))\right]^2}{(7.25\ \mathrm{ft})^2 \times \left[(3 \times (6.5014\ \mathrm{kipft/ft})) + (2 \times (-0.60605\ \mathrm{kip/ft}) \times (7.25\ \mathrm{ft}))\right]}$$

$$p = 0.21897\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times \left[(2 \times (6.5014\ \mathrm{kipft/ft})) + ((-0.60605\ \mathrm{kip/ft}) \times (7.25\ \mathrm{ft}))\right]}{(7.25\ \mathrm{ft})^2}$$

$$s = 0.98272\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.021\ \mathrm{ft})}{2}$$

$$p_a = 0.37657\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.21897\ \mathrm{kip/ft^2}\right)}{\left(0.37657\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.58148$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.25\ \mathrm{ft})$$

$$p_s = 1.0875\ \mathrm{kip/ft^2}$$

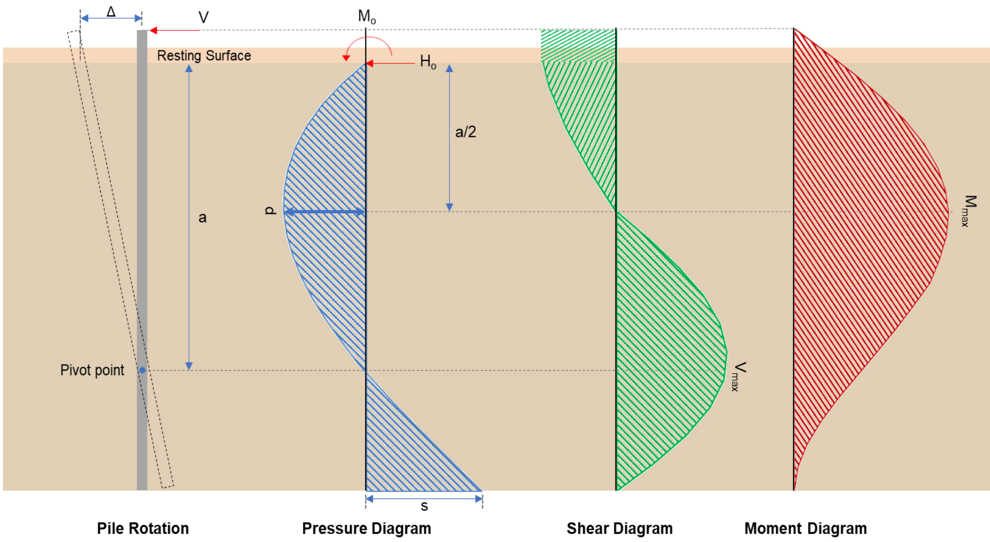
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.98272\ \mathrm{kip/ft^2}\right)}{\left(1.0875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.90365$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.343\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -1.01\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.239\ \text{kipft}) + ((-6.343\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 11.025\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.025\ \text{kipft/ft})}{(-1.01\ \text{kip/ft})}$$

$$E = 10.916\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.025\ \text{kipft/ft}) \times (7.25\ \text{ft})) + \left(3 \times (-1.01\ \text{kip/ft}) \times (7.25\ \text{ft})^2\right)}{(6 \times (11.025\ \text{kipft/ft})) + (4 \times (-1.01\ \text{kip/ft}) \times (7.25\ \text{ft}))}$$

$$a = 5.0187\ \text{ft}$$

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

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

$$V_{max} = ((-1.01\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.916\ \text{ft})}{(7.25\ \text{ft})} + 3\right) \times \left(\frac{(5.0187\ \text{ft})}{(7.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.916\ \text{ft})}{(7.25\ \text{ft})} + 2\right) \times \left(\frac{(5.0187\ \text{ft})}{(7.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 13.428\ \text{kip}$$

$$V_{max} = 13.428 \text{ kip}$$

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

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

$$M_{max} = ((-1.01 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(10.916 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0187 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.916 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0187 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.916 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0187 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 46.021 \text{ kipft}$$

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,

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

$$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (7.25 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft})^2\right)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.8333 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(7.25 \text{ ft})} \right) \times \left(\frac{(4.8333 \text{ ft})}{(7.25 \text{ ft})} - 1 \right) \times \left(\frac{(4.8333 \text{ ft})}{(7.25 \text{ ft})} \right)^2$$

$$V_{max} = 0.00031237 \text{ kip}$$

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

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

$$M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.8333 \text{ ft})}{2 \times (7.25 \text{ ft})}^3 \right) + \left(3 \times \frac{(4.8333 \text{ ft})}{2 \times (7.25 \text{ ft})}^4 \right) \right]$$

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

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,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.068\text{ kip})}{(0.65)\times(0.8)} - \left(0.85\times(2.5\text{ ksi})\times\left(2304\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85\times(2.5\text{ ksi}))}, \left(0.08\times\left(2304\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.162\text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.162\text{ in}^2\right), \left(0.0018\times\left(2304\text{ in}^2\right)\right)\right]$$

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

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

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

$$Ratio = \frac{(13.068 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0048849$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.068 \text{ kip} \rightarrow 13068 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13068 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.23 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.23 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.23 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.23 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.23 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.428$ kip - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(13.428 \text{ kip})}{(111.23 \text{ kip})}$$

$$Ratio = 0.12072$$

Status: **PASS**
Ratio: **0.120**

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:** $M_{max} = 46.021 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(46.021 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.18438$$	Status: **PASS** Ratio: **0.180**
	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**