Project Details

Project Name: MTSOLAR_J0DK8KAI349A

Location: 518 Turkey St, North Berwick, ME 03906, USA

Unique ID: 2P-17-6TOP-SD-24-L-4Hx5W-H883

Dealer: ______________________________

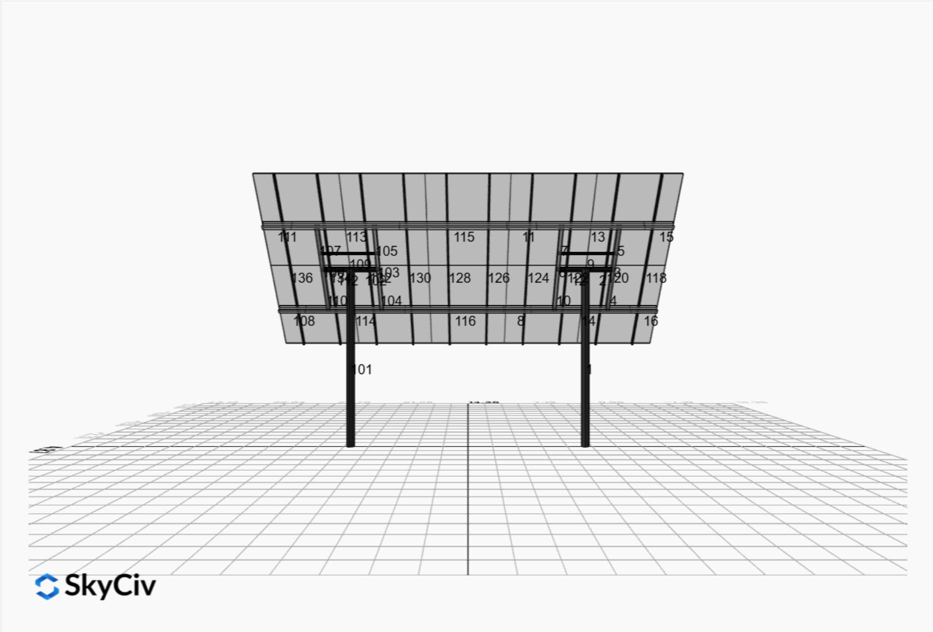
Date: Fri Nov 01 2024

Number of Modules: 20

Number of Poles: 2

Date Sold:

____



Array Dimensions N/S	15.17 ft
Array Dimensions E/W	28.75 ft
Winter Tilt Angle	50
Front Edge Clearance	7 ft

MT Solar Bill of Materials (2P-17-6TOP-SD-24-L-4Hx5W-H883)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-SD	H-Frame Assembly-SD	2
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-57	57IN SD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	5

Rail Bill of Materials

Part	Qty
Rails (180in)	10
Rail Attachment	20
Module Mid Clamp	30
Module End Clamp	20
Ground Lug	5

Site Details:

Site Address: 518 Turkey St, North Berwick, ME 03906, USA

Array Specification

Duty Classification:	SD
Module Width:	45.00 in
Module Length:	68.00in
Number of Rows:	4
Number of Columns:	5
Total Number of Modules:	20
Winter Tilt Angle:	50
Front Edge Clearance:	7
Total Array Height at Tilt:	18.62 ft
Total Frame Length:	28.50 ft
Frame Weight:	1976 lbs
Array Dimensions N/S:	15.17 ft
Array Dimensions E/W:	28.75 ft
Rail Length:	182.00 in
Rail Spacing:	2.88 ft

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	12.81 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 6.75 ft
Foundation Volume:	8.000 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	518 Turkey St, North Berwick, ME 03906, USA
Wind Speed:	105 mph
Snow Load:	70 psf

Design Disclaimer

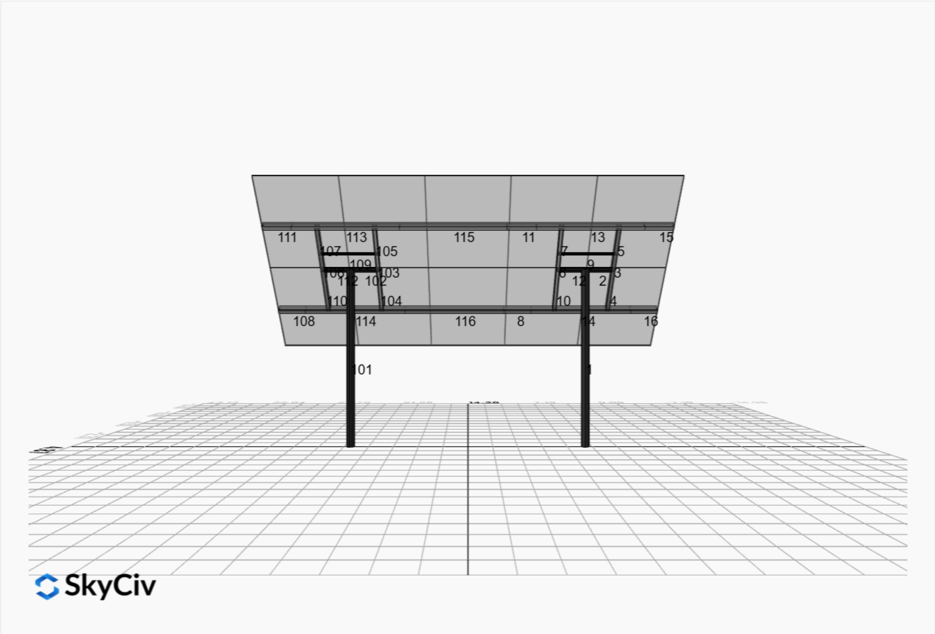
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

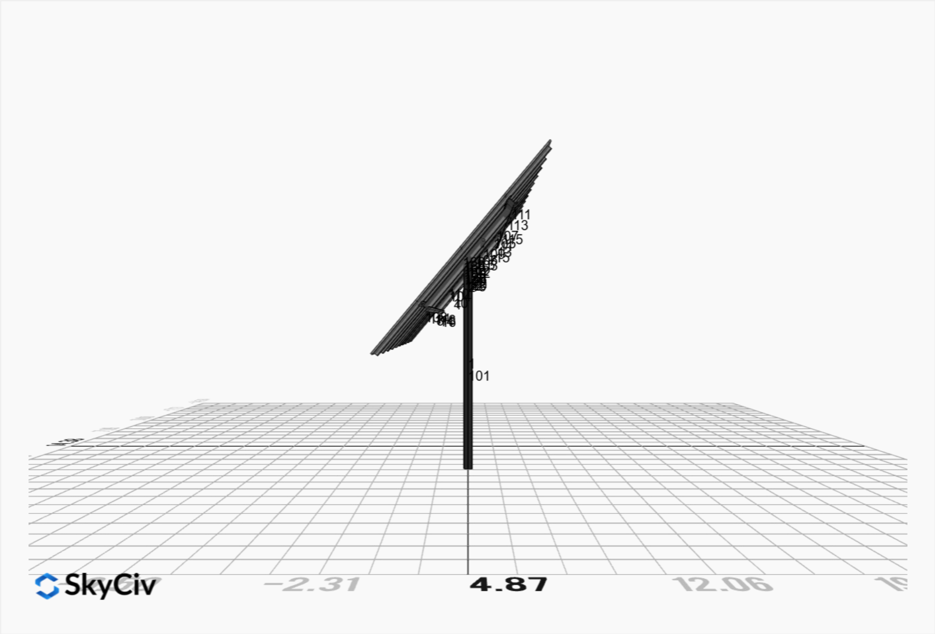
AutoDesigner Input

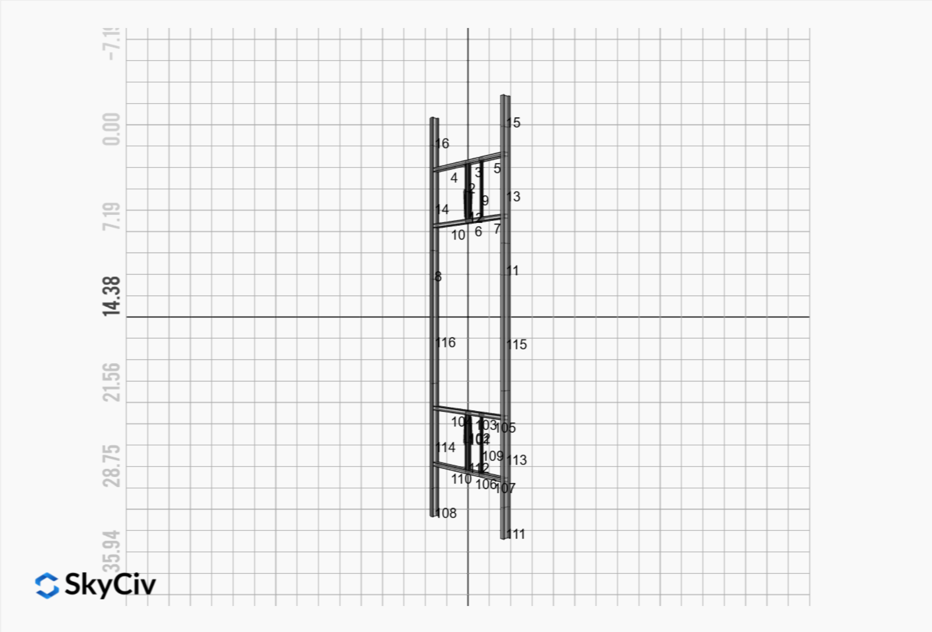
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"jk","designer_email":"Adrianne@srsolarnh.com","designer_phone":"6035404243","project_id":"MTSOLAR_J0DK8KAI349A","site_address":"518 Turkey St, North Berwick, ME 03906, USA","module_width":45,"module_length":68,"number_rows":"4","number_columns":5,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":50,"ground_clearance":7,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}

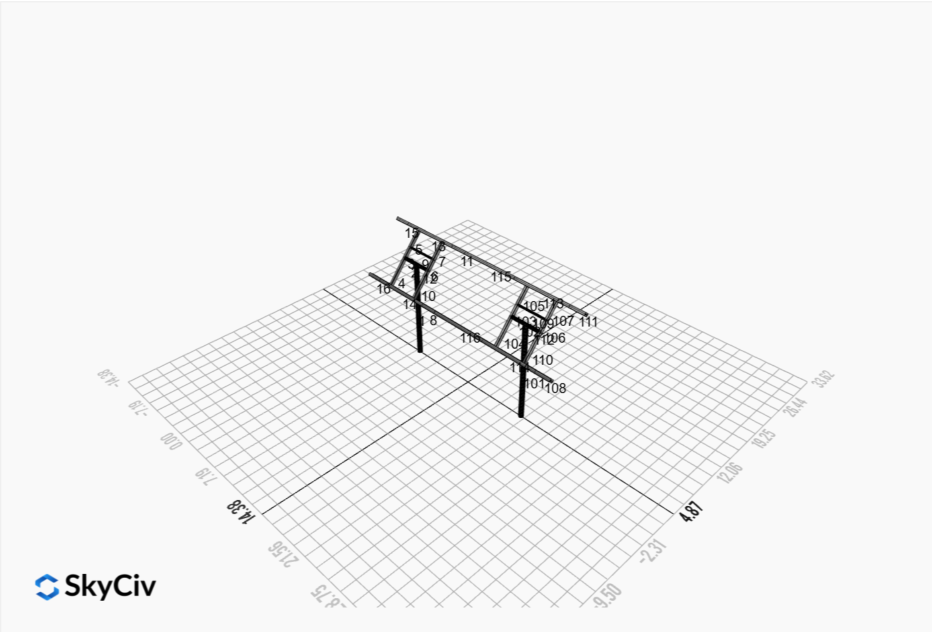
Design Notes:

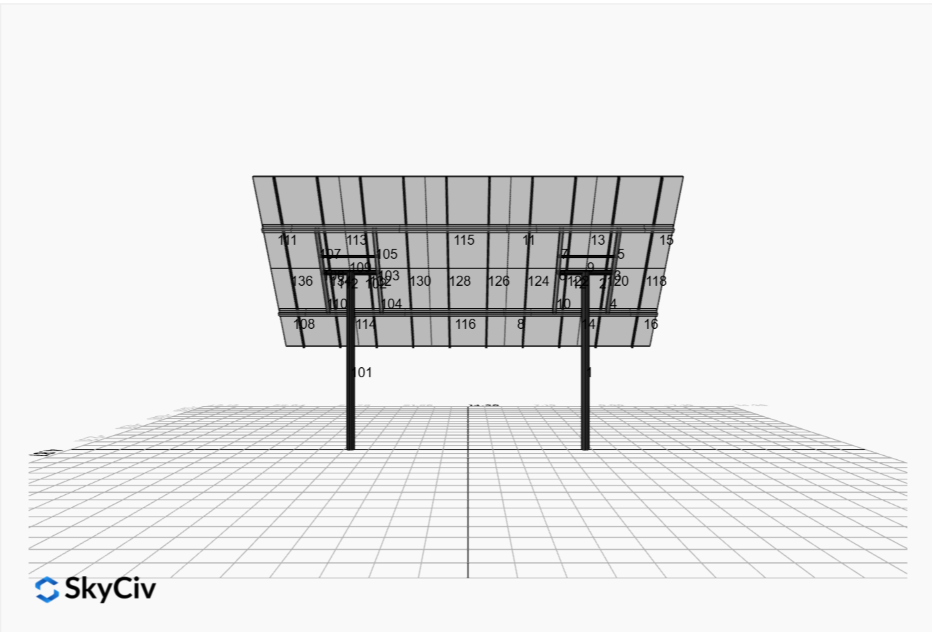
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only



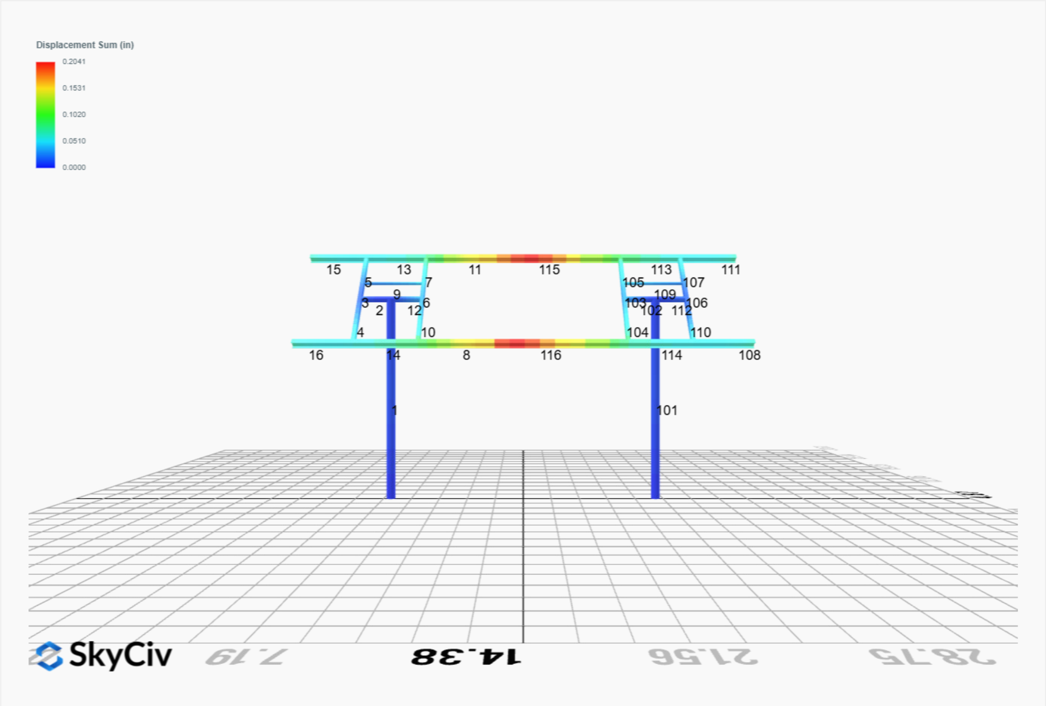


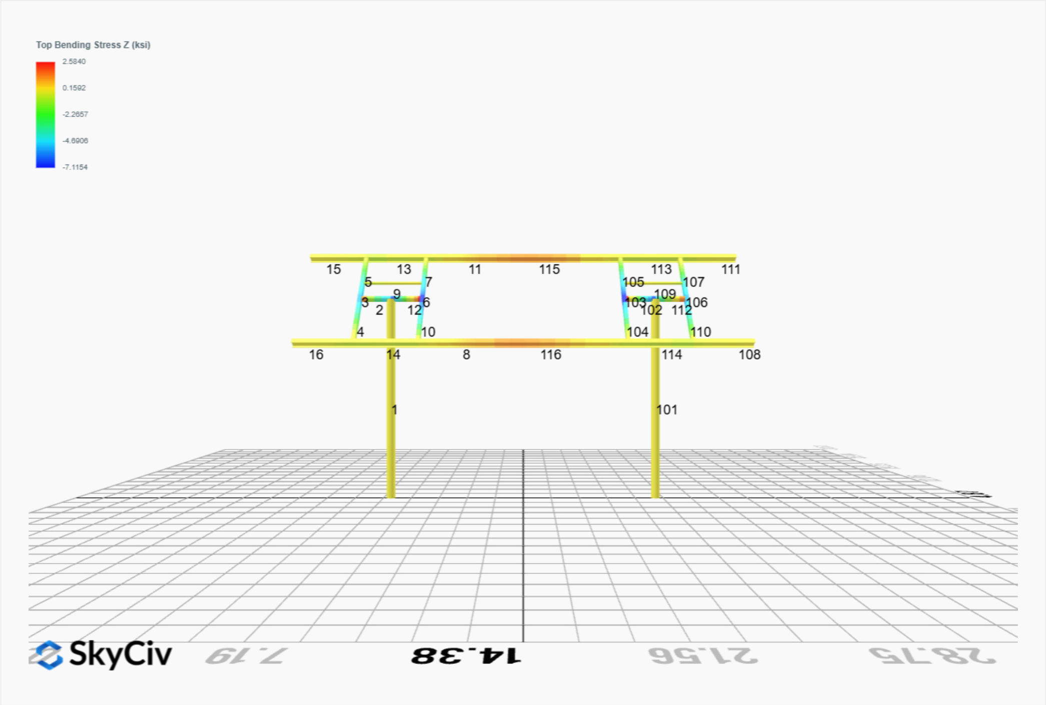

0.00
7.19
14.38
21.56
28.75
16
15
4
3
5
13
14
10
6
7
8
11
116
115
114
113
110
106
107
108
111
SkyCiv

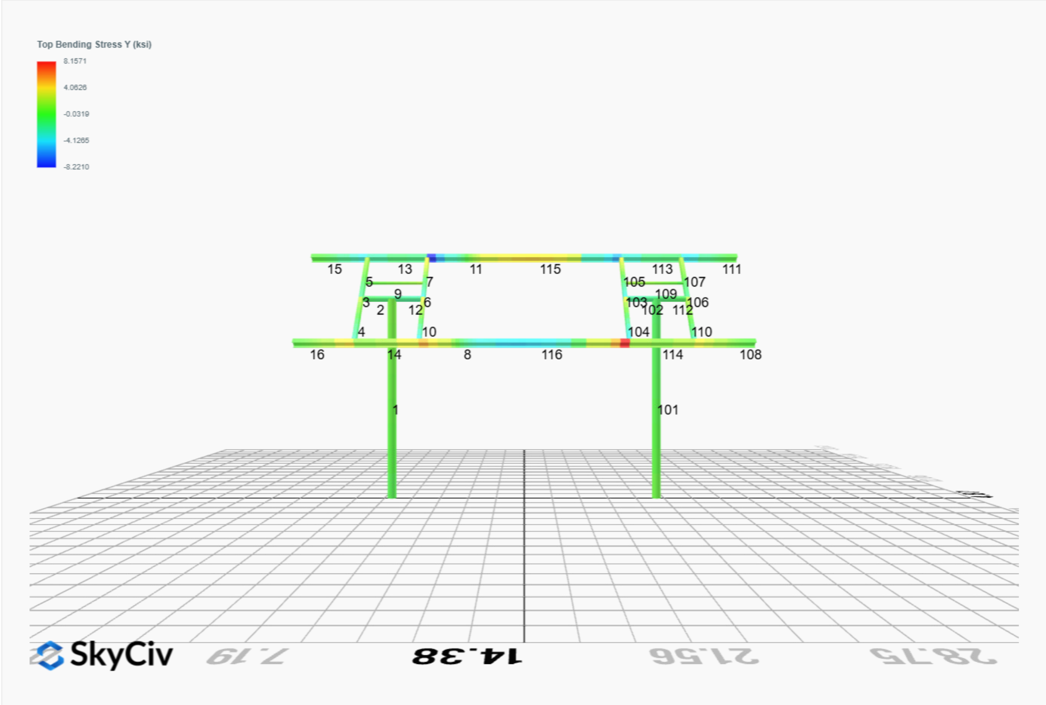

0.00
7.19
14.38
21.56
4.87
12.06
19.25
26.44
-2.31
-9.50
115
116
111
108
SkyCiv

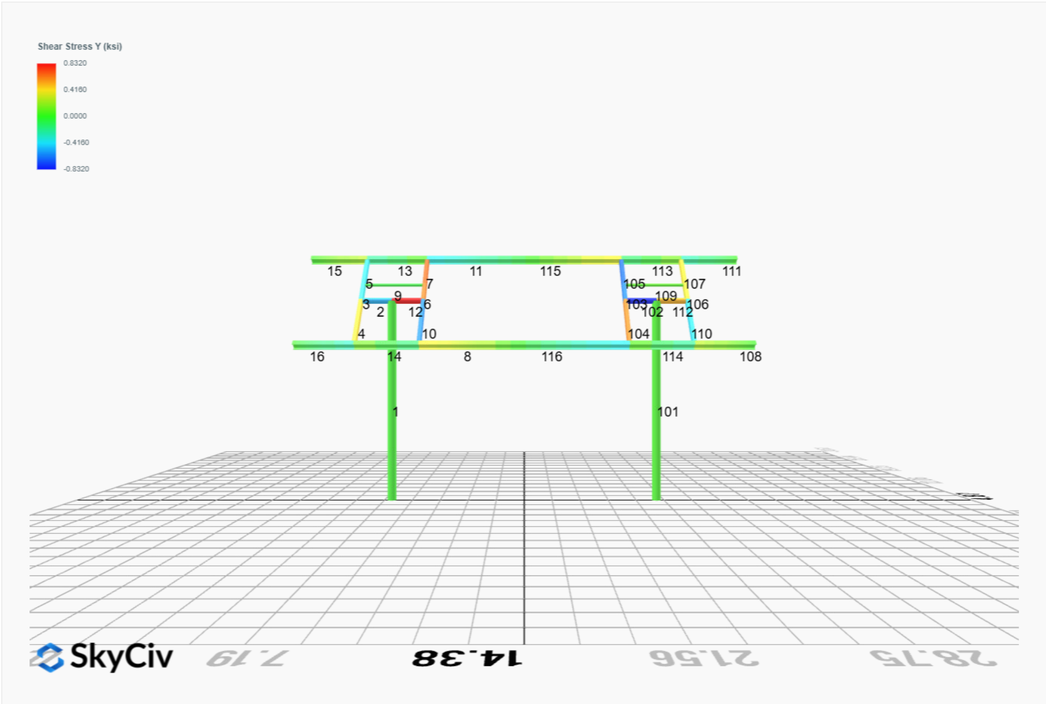

111
113
115
11
13
15
107
105
136
130
128
126
124
120
118
110
104
108
114
116
8
16
101
SkyCiv

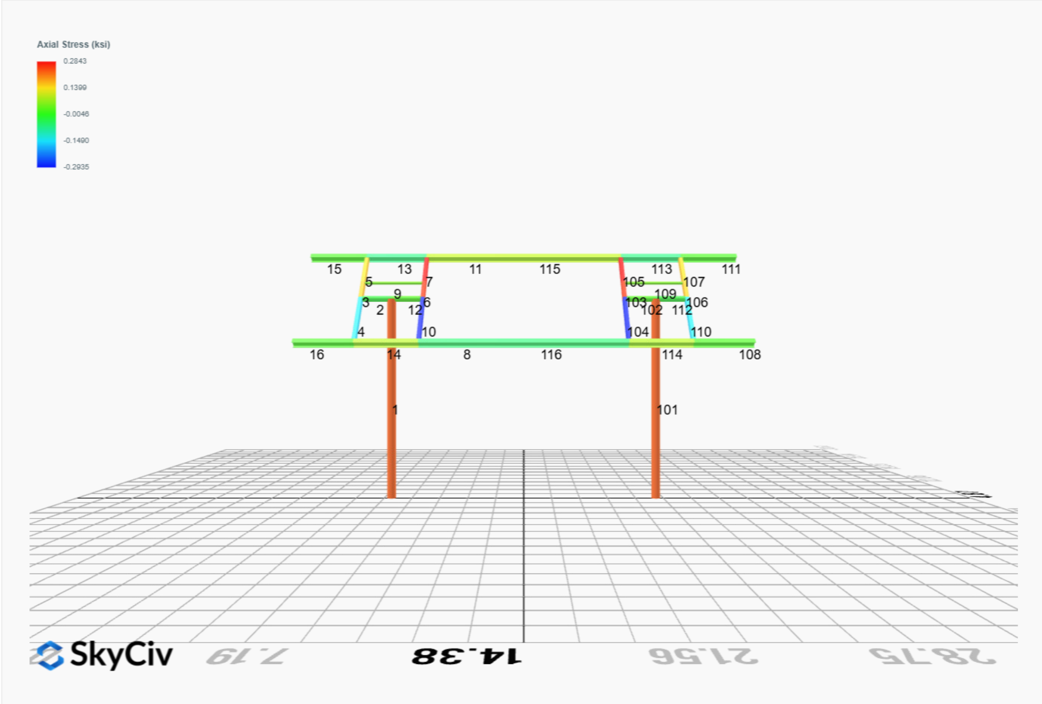
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
8.1571
4.0626
-0.0319
-4.1265
-8.2210
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
SkyCiv
7.19
14.38
21.56
28.75


Shear Stress Y (ksi)
0.8320
0.4160
0.0000
-0.4160
-0.8320
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
SkyCiv
7.19
14.38
21.56
28.75


Axial Stress (ksi)
0.2843
0.1399
-0.0046
-0.1490
-0.2935
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
14.38
21.56
28.75
7.19

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	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 2. D + L	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 3. D + (S or Lr or R)	0.0000	3.9352	0.0977	0.3995	-0.1209	0.0241
ULS: 3. D + (S or Lr or R)	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.4005	0.0824	0.3370	-0.1019	0.0219
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 5b. D + 0.7E	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.4005	0.0824	0.3370	-0.1019	0.0219
ULS: 8. 0.6D + 0.7E	0.0000	1.0779	0.0219	0.0896	-0.0270	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3710	3.7860	0.1255	0.5007	-0.4233	31.0796
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3710	-0.1930	-0.0523	-0.2004	0.3337	-29.6964
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7782	4.8926	0.1491	0.6005	-0.3856	23.3199
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.4005	0.0824	0.3370	-0.1019	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7782	1.9084	0.0158	0.0747	0.1822	-22.2620
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.4005	0.0824	0.3370	-0.1019	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7782	3.2886	0.1033	0.4129	-0.3288	23.3135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7782	0.3044	-0.0301	-0.1129	0.2390	-22.2684
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.7965	0.0366	0.1493	-0.0451	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3710	3.0674	0.1109	0.4410	-0.4053	31.0733
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0779	0.0219	0.0896	-0.0270	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3710	-0.9116	-0.0669	-0.2601	0.3517	-29.7026
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0779	0.0219	0.0896	-0.0270	0.0093

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	7.2356
Shear X	-3.9516
Shear Z	0.2221
Moment X	0.8896
Moment Y (Twist)	0.7193
Moment Z	53.0346

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	4.8926
Shear X	-2.3710
Shear Z	0.1491
Moment X	0.6005
Moment Y (Twist)	0.4233
Moment Z	31.0796

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 2. D + L	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 3. D + (S or Lr or R)	-0.0000	3.9352	-0.0977	-0.3995	0.1209	0.0241
ULS: 3. D + (S or Lr or R)	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.4005	-0.0824	-0.3370	0.1019	0.0219

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 5b. D + 0.7E	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.4005	-0.0824	-0.3370	0.1019	0.0219
ULS: 8. 0.6D + 0.7E	-0.0000	1.0779	-0.0219	-0.0896	0.0270	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3710	3.7860	-0.1255	-0.5007	0.4233	31.0796
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3710	-0.1930	0.0523	0.2004	-0.3337	-29.6964
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7782	4.8926	-0.1491	-0.6005	0.3856	23.3200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	3.4005	-0.0824	-0.3370	0.1019	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7782	1.9084	-0.0158	-0.0747	-0.1822	-22.2620
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	3.4005	-0.0824	-0.3370	0.1019	0.0219
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7782	3.2886	-0.1033	-0.4129	0.3288	23.3135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7782	0.3044	0.0301	0.1129	-0.2390	-22.2684
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.7965	-0.0366	-0.1493	0.0451	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3710	3.0674	-0.1109	-0.4410	0.4053	31.0733
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.0779	-0.0219	-0.0896	0.0270	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3710	-0.9116	0.0669	0.2601	-0.3517	-29.7026
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.0779	-0.0219	-0.0896	0.0270	0.0093

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	7.2356
Shear X	-3.9516
Shear Z	-0.2221
Moment X	-0.8898
Moment Y (Twist)	0.7189
Moment Z	53.0355

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	4.8926
Shear X	-2.3710
Shear Z	-0.1491
Moment X	-0.6005
Moment Y (Twist)	0.4233
Moment Z	31.0796

Project Details

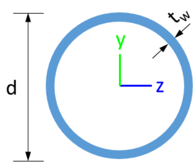
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	MTSOLAR_J0DK8KAI349A
Unit System:	imperial

Design Input Information

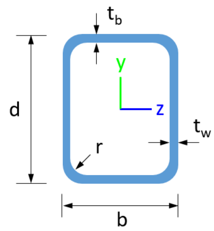
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

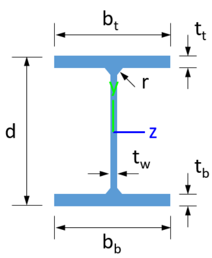
Section Dimensions



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					
8	6in Pipe Sch 80	6.63	0.43					



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

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
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
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

Member Properties

Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	8	26.90	26.90	12.81	-	300	200	1
2	4	2.00	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.70,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.20,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
8	18	1.33	1.33	2.05	1.29,1.29,1.29,1.29,1.29,1.29,1.28,1.29,1.27,1.29,1.28,1.29,1.28,1.29,1.29,1.29,1.39,1.29,1.28,1.29,1.27,1.29,1.28,1.29,1.28,1.29	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	1.33	1.33	2.05	1.30,1.30,1.30,1.30,1.30,1.30,1.29,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.29,1.30,1.39,1.30,1.29,1.30,1.27,1.30,1.28,1.30,1.28,1.30	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.23,1.23,1.23,1.23,1.23,1.23,1.25,1.23,1.27,1.23,1.25,1.23,1.26,1.23,1.24,1.23,1.11,1.23,1.25,1.23,1.28,1.23,1.25,1.23,1.26,1.23	300	200	1
14	18	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.24,1.22,1.26,1.22,1.24,1.22,1.25,1.22,1.23,1.22,1.10,1.22,1.24,1.22,1.27,1.22,1.24,1.22,1.25,1.22	300	200	1
15	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	8	26.90	26.90	12.81	-	300	200	1
102	4	1.30	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.20,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
106	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18	300	200	1
107	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
108	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
109	1	2.60	2.60	4.00	-	300	200	1
110	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.70,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
111	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
112	4	2.00	1.30	2.00	-	300	200	1

113	18	4.88	4.00	7.50	1.23,1.23,1.23,1.23,1.23,1.23,1.25,1.23,1.27,1.23,1.25,1.23,1.26,1.23,1.24,1.23,1.12,1.23,1.25,1.23,1.27,1.23,1.25,1.23,1.26,1.23	300	200	1
114	18	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.24,1.22,1.26,1.22,1.24,1.22,1.25,1.22,1.23,1.22,1.10,1.22,1.24,1.22,1.27,1.22,1.24,1.22,1.25,1.22	300	200	1
115	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09	300	200	1
116	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.08,1.09,1.08,1.09,1.08,1.09	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	87.79	62.23	62.23	113.47	113.47
2	142.83	140.22	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	115.40	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	115.40	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	19.60	6.45	30.09	45.74
14	120.60	84.03	19.42	6.45	30.09	45.74
15	120.60	96.18	23.36	6.45	30.09	45.74
16	120.60	96.18	23.36	6.45	30.09	45.74
101	378.22	87.79	62.23	62.23	113.47	113.47
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	6.26	29.14	16.61
104	79.65	72.01	10.99	6.26	29.14	16.61
105	79.65	73.44	10.99	6.26	29.14	16.61
106	79.65	74.02	10.99	6.26	29.14	16.61
107	79.65	73.44	10.99	6.26	29.14	16.61
108	120.60	96.18	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	6.26	29.14	16.61
111	120.60	96.18	23.36	6.45	30.09	45.74
112	142.83	140.22	16.17	16.17	42.85	42.85
113	120.60	84.03	19.64	6.45	30.09	45.74
114	120.60	84.03	19.37	6.45	30.09	45.74
115	120.60	84.26	19.15	6.45	30.09	45.74
116	120.60	84.26	19.14	6.45	30.09	45.74

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	0.082	0.852	0.031	0.035	0.002	0.904	#13	0.735	Not Required	Pass
2	0.001	0.264	0.216	0.066	0.044	0.462	#13	0.079	Not Required	Pass
3	0.011	0.537	0.074	0.053	0.014	0.576	#13	0.044	Not Required	Pass
4	0.009	0.534	0.112	0.054	0.022	0.609	#13	0.078	Not Required	Pass

4	0.009	0.534	0.112	0.054	0.022	0.609	#13	0.078	Not Required	Pass
5	0.010	0.333	0.092	0.053	0.016	0.340	#13	0.073	Not Required	Pass
6	0.016	0.654	0.159	0.066	0.036	0.747	#13	0.044	Not Required	Pass
7	0.016	0.406	0.203	0.065	0.039	0.432	#13	0.073	Not Required	Pass
8	0.003	0.126	0.088	0.038	0.014	0.193	#21	0.088	Not Required	Pass
9	0.001	0.042	0.074	0.003	0.003	0.105	#13	0.198	Not Required	Pass
10	0.016	0.651	0.198	0.065	0.036	0.709	#13	0.078	Not Required	Pass
11	0.003	0.125	0.088	0.038	0.014	0.192	#21	0.088	Not Required	Pass
12	0.001	0.381	0.260	0.092	0.051	0.627	#13	0.052	Not Required	Pass
13	0.005	0.098	0.277	0.052	0.019	0.330	#21	0.265	Not Required	Pass
14	0.004	0.100	0.273	0.052	0.019	0.324	#21	0.177	Not Required	Pass
15	0.000	0.021	0.041	0.016	0.006	0.059	#21	Not Required	Not Required	Pass
16	0.000	0.021	0.041	0.016	0.006	0.059	#21	Not Required	Not Required	Pass
101	0.082	0.852	0.031	0.035	0.002	0.904	#13	0.735	Not Required	Pass
102	0.001	0.381	0.260	0.092	0.051	0.627	#13	0.052	Not Required	Pass
103	0.016	0.654	0.159	0.066	0.036	0.747	#13	0.044	Not Required	Pass
104	0.016	0.651	0.198	0.065	0.036	0.709	#13	0.078	Not Required	Pass
105	0.016	0.406	0.203	0.065	0.039	0.432	#13	0.073	Not Required	Pass
106	0.011	0.537	0.074	0.053	0.014	0.577	#13	0.044	Not Required	Pass
107	0.010	0.333	0.092	0.053	0.016	0.340	#13	0.073	Not Required	Pass
108	0.000	0.021	0.041	0.016	0.006	0.059	#21	Not Required	Not Required	Pass
109	0.001	0.042	0.074	0.003	0.003	0.105	#13	0.198	Not Required	Pass
110	0.009	0.533	0.112	0.054	0.022	0.609	#13	0.078	Not Required	Pass
111	0.000	0.021	0.041	0.016	0.006	0.059	#21	Not Required	Not Required	Pass
112	0.001	0.264	0.216	0.066	0.044	0.462	#13	0.079	Not Required	Pass
113	0.005	0.098	0.276	0.052	0.019	0.330	#21	0.177	Not Required	Pass
114	0.004	0.100	0.273	0.052	0.019	0.324	#21	0.265	Not Required	Pass
115	0.004	0.197	0.164	0.038	0.014	0.329	#21	0.321	Not Required	Pass
116	0.003	0.198	0.163	0.038	0.014	0.330	#21	0.321	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comresssion)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis

V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

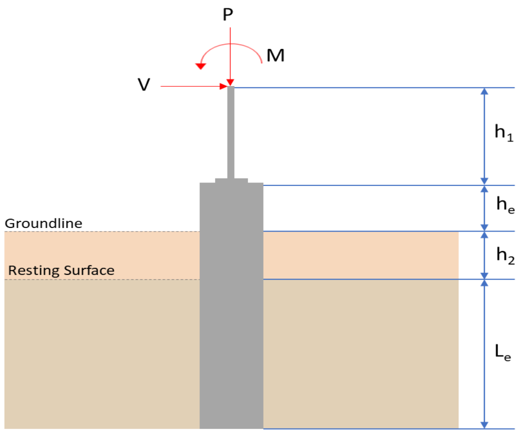
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 6.75\ \text{ft}$ - 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)	4.893	7.236
V_x (kip)	-2.371	-3.952
V_z (kip)	0.149	0.222
M_x (kipft)	0.601	0.890
M_z (kipft)	31.080	53.035

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,

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

$$H_o = \frac{(-2.371\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.37755\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.08\ \mathrm{kipft}) + ((-2.371\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.949\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.3222\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.149\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.023726\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.601\ \mathrm{kipft}) + ((0.149\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.095701\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 2.2107\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.3222\ \mathrm{ft}), (2.2107\ \mathrm{ft})]$$

$$L_{e,req} = 6.322\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.322\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.93659$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48\ \mathrm{in}) \times (48\ \mathrm{in})$$

$$A = 16\ \mathrm{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.893\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.30581\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.30581\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.15291$$

Status: **PASS**
Ratio: **0.150**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.37755\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.949\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.949\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.37755\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (4.949\ \mathrm{kipft/ft})) + (4 \times (-0.37755\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6438\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.949\ \mathrm{kipft/ft})) + (3 \times (-0.37755\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\,2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (4.949\ \mathrm{kipft/ft})) + (2 \times (-0.37755\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.24926\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.949\ \mathrm{kipft/ft})) + ((-0.37755\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.96785\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6438\ \mathrm{ft})}{2}$$

$$p_a = 0.34828\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.24926 \text{ kip/ft}^2\right)}{\left(0.34828 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.71568$$

Status: **PASS**
Ratio: **0.720**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.96785 \text{ kip/ft}^2\right)}{\left(1.0125 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.95591$$

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = 0.023726 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.095701 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.095701 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (0.023726 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.095701 \text{ kipft/ft})) + (4 \times (0.023726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7966 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.095701 \text{ kipft/ft})) + (3 \times (0.023726 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.095701 \text{ kipft/ft})) + (2 \times (0.023726 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.020196 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.095701 \text{ kipft/ft})) + ((0.023726 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.046295 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7966 \text{ ft})}{2}$$

$$p_a = 0.35975 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.020196 \text{ kip/ft}^2\right)}{\left(0.35975 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.056138$$

Status: **PASS**
Ratio: **0.060**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

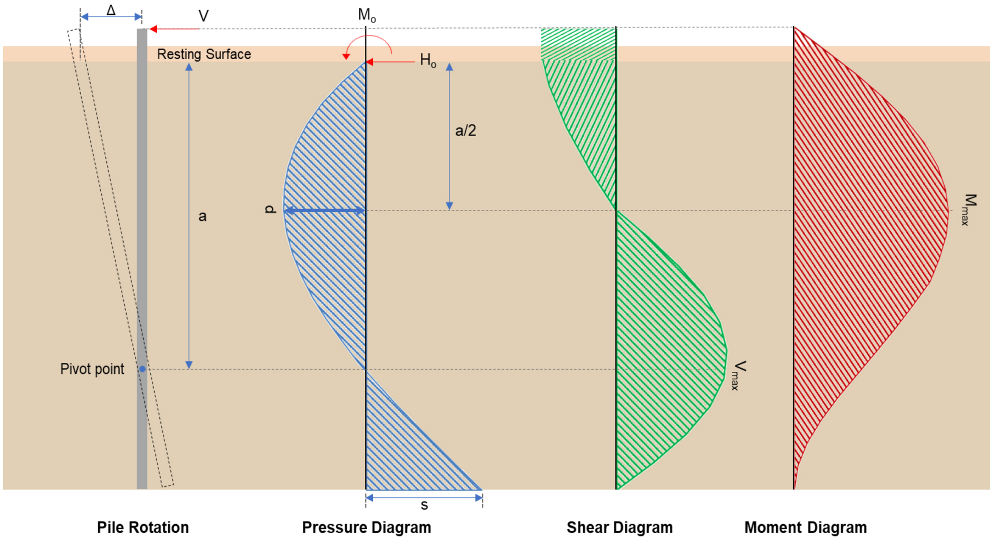
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.046295\ \mathrm{kip/ft^2})}{(1.0125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.045723$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.952\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.6293\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.035\ \mathrm{kipft}) + ((-3.952\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.4451\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.4451\ \mathrm{kipft/ft})}{(-0.6293\ \mathrm{kip/ft})}$$

$$E = 13.42\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4451\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.6293\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.4451 \text{ kipft/ft})) + (4 \times (-0.6293 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6413 \text{ ft}$$

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

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

$$V_{max} = ((-0.6293 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.42 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.6413 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (13.42 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.6413 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 10.517 \text{ kip}$$

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

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

$$M_{max} = ((-0.6293 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(13.42 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6413 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (13.42 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.6413 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (13.42 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.6413 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 33.95 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.222 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.03535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.89 \text{ kipft}) + ((0.222 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14172 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14172 \text{ kipft/ft})}{(0.03535 \text{ kip/ft})}$$

$$E = 4.009 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14172 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (0.03535 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.14172 \text{ kipft/ft})) + (4 \times (0.03535 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7975 \text{ ft}$$

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

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

$$V_{max} = ((0.03535\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.009\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.7975\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.009\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.7975\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.24258\ \text{kip}$$

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

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

$$M_{max} = ((0.03535\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(4.009\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.7975\ \text{ft})}{2 \times (6.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (4.009\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.7975\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.009\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.7975\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.73336\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.236\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.356\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.356\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\, \frac{\pi\, d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625\ \text{in})^2}{4}$$

$$A_{st} = 4.2951\ \text{in}^2$$

$Ratio$ - Capacity

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{(4.1472\ \text{in}^2)}{(4.2951\ \text{in}^2)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.236\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0027049$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.236 \text{ kip} \rightarrow 7236 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7236 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.45 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.45 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.45 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.45 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.72 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.517 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(10.517\ \text{kip})}{(110.72\ \text{kip})}$$ $$Ratio = 0.094986$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $V_{max} = 0.24258\ \text{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.24258\ \text{kip})}{(110.72\ \text{kip})}$$ $$Ratio = 0.0021908$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f'_{ck}\ S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(18432\ \text{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \text{kipft}$$ 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:** $M_{max} = 33.95\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(33.95\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.13602$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 0.73336\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(0.73336 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0029382$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

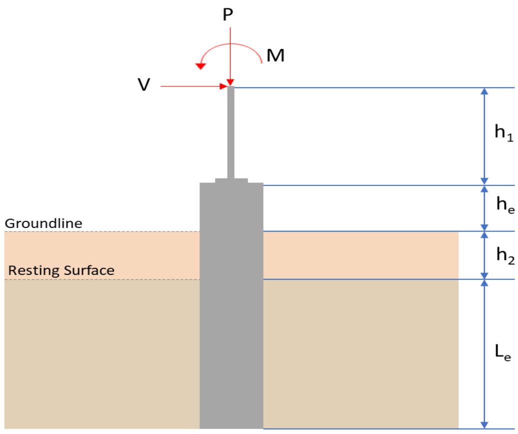
SkyCiv Foundation Design

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
$L = 6.75 \text{ ft}$ - 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)	4.893	7.236
V_x (kip)	-2.371	-3.952
V_z (kip)	-0.149	-0.222
M_x (kipft)	-0.601	-0.890
M_z (kipft)	31.080	53.035

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,

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

$$H_o = \frac{(-2.371 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.37755 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.08\ \text{kipft}) + ((-2.371\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.949\ \text{kipft/ft}$$

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:
$L_{e,x} = 6.3222\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.149\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.023726\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.601\ \text{kipft}) + ((-0.149\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.095701\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.7315\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.3222\ \text{ft}), (1.7315\ \text{ft})]$$

$$L_{e,req} = 6.322\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.322\ \text{ft})}{(6.75\ \text{ft})}$$

$$Ratio = 0.93659$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\ D$$

$$A = (48\ \text{in}) \times (48\ \text{in})$$

$$A = 16\ \text{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.893 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.30581 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.30581 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.15291$$

Status: **PASS**
Ratio: **0.150**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.37755 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.949 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.949 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.37755 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (4.949 \text{ kipft/ft})) + (4 \times (-0.37755 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6438 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.949 \text{ kipft/ft})) + (3 \times (-0.37755 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.949 \text{ kipft/ft})) + (2 \times (-0.37755 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.24926 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.949 \text{ kipft/ft})) + ((-0.37755 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.96785 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6438 \text{ ft})}{2}$$

$$p_a = 0.34828 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.24926\ \mathrm{kip/ft^2}\right)}{\left(0.34828\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.71568$$

Status: **PASS**
Ratio: **0.720**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.96785\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.95591$$

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = -0.023726\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.095701\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.095701\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.023726\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.095701\ \mathrm{kipft/ft})) + (4 \times (-0.023726\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.7966\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.095701\ \mathrm{kipft/ft})) + (3 \times (-0.023726\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (0.095701\ \mathrm{kipft/ft})) + (2 \times (-0.023726\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = -0.0047278\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.095701\ \mathrm{kipft/ft})) + ((-0.023726\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.0041152\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.7966\ \mathrm{ft})}{2}$$

$$p_a = 0.35975\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0047278\ \mathrm{kip/ft^2}\right)}{\left(0.35975\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.013142$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

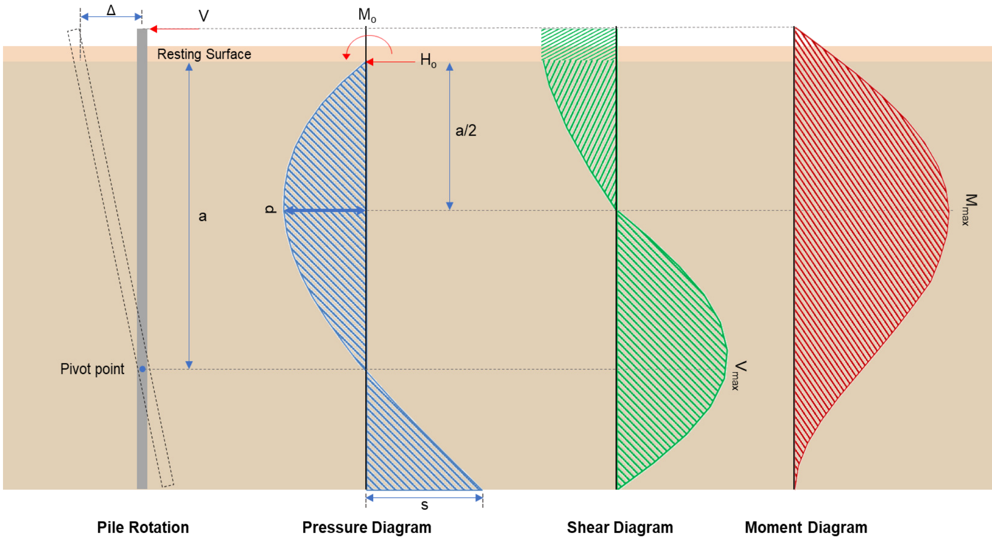
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0041152\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0040644$$

Status: **PASS**
Ratio: **0.000**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.952\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.6293\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.035\ \mathrm{kipft}) + ((-3.952\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.4451\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.4451\ \mathrm{kipft/ft})}{(-0.6293\ \mathrm{kip/ft})}$$

$$E = 13.42\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4451\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.6293\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{\ldots}{(6 \times (8.4451\ \mathrm{kipft/ft})) + (4 \times (-0.6293\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6413\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.6293\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.42\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6413\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (13.42\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6413\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.517\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.6293\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.75\ \mathrm{ft})) \times \left[\left(\frac{(13.42\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + \frac{(4.6413\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (13.42\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6413\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (13.42\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6413\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 33.95\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.222\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.03535\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.89\ \mathrm{kipft}) + ((-0.222\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.14172\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14172\ \mathrm{kipft/ft})}{(-0.03535\ \mathrm{kip/ft})}$$

$$E = 4.009\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14172\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.03535\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.14172\ \mathrm{kipft/ft})) + (4 \times (-0.03535\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.7975\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.03535\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.009\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.7975\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.009\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.7975\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.24258\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.03535\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.75\ \mathrm{ft})) \times \left[\left(\frac{(4.009\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + \frac{(4.7975\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (4.009\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.7975\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.009\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.7975\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.73336\ \mathrm{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.236\ \mathrm{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \mathrm{ksi}) \times (2304\ \mathrm{in}^2))}{(60\ \mathrm{ksi}) - (0.85 \times (2.5\ \mathrm{ksi}))}, (0.08 \times (2304\ \mathrm{in}^2))\right]$$

$$A_{st,required} = -84.356\ \mathrm{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.356\ \mathrm{in}^2), (0.0018 \times (2304\ \mathrm{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\, \frac{\pi\, d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625\ \mathrm{in})^2}{4}$$

$$A_{st} = 4.2951\ \mathrm{in}^2$$

$Ratio$ - Capacity

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{(4.1472\ \mathrm{in}^2)}{(4.2951\ \mathrm{in}^2)}$$

$$Ratio = 0.96556$$

Status: PASS
Ratio: 0.970

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 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

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

$$Ratio = \frac{(7.236 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0027049$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.236 \text{ kip} \rightarrow 7236 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7236 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.45 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.45 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.45 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.45 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.72 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.517 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.517 \text{ kip})}{(110.72 \text{ kip})}$$

$$Ratio = 0.094986$$

Status: **PASS**
Ratio: **0.090**

Considering z-direction:

$V_{max} = 0.24258 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.24258 \text{ kip})}{(110.72 \text{ kip})}$$

$$Ratio = 0.0021908$$

Status: **PASS**
Ratio: **0.000**

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:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$

$$\phi M_{n,1} = 249.600 kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

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:

$M_{max} = 33.95 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$Ratio = \frac{(33.95 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13602$$

Status: **PASS**
Ratio: **0.140**

Considering z-direction:

$M_{max} = 0.73336 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

	$$Ratio = \frac{(0.73336 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0029382$$	Status: **PASS** Ratio: **0.000**