Your Project Calculations

Project Name: CORiosRevA

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=CORiosRevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=FLKm8zF1koylmggfCnw1fKHxETrcTeRdfvTZeD2h3edwj2p1OP014DtjA3lrXXKz

Array Specification

Product:	Beam
Unique ID:	2P-22.5-8TOP-HD-45-L-4Hx6W-J97K
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	75.08in
Number of Rows:	4
Number of Columns:	6
Total Number of Modules:	24
Desired Tilt Angle:	37
Front Edge Clearance:	5
Total Array Height at Tilt:	13.30 ft
Total Frame Length:	37.50 ft
Frame Weight:	2166 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	38.04 ft
Rail Length:	166.40 in
Rail Spacing:	3.13 ft
Rail Check:	Not Checked

Support Specifications

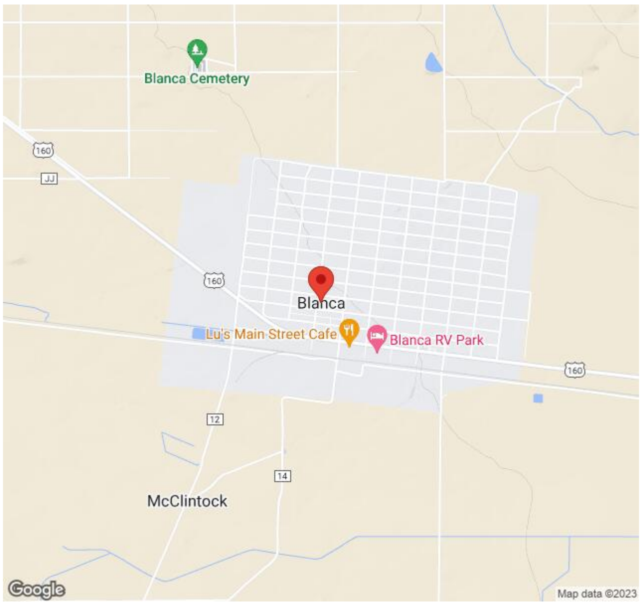
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	9.17 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft
Foundation Volume:	7.407 y^3
Foundation Result:	PASSED
Mount Twist:	1.382268 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Blanca, CO 81123, USA
Wind Speed:	105 mph
Snow Load:	35 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012701 ksf



Design Disclaimer

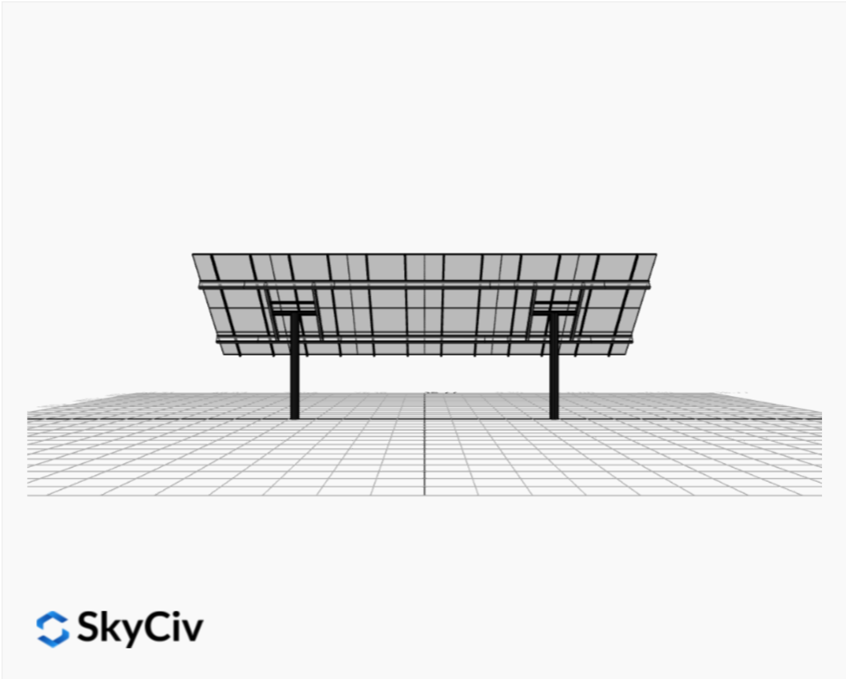
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

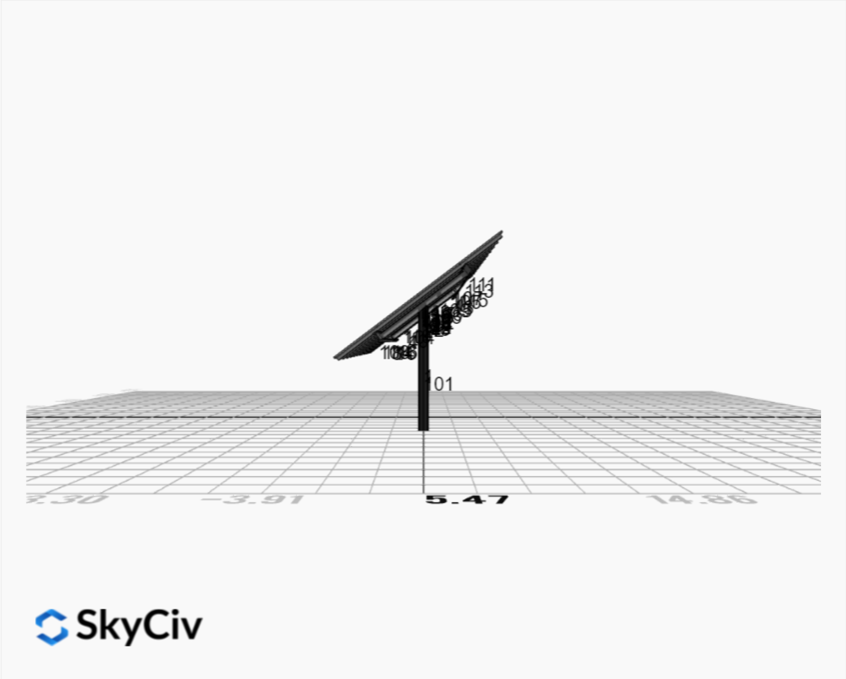
AutoDesigner Input

{"product_type":"Beam","project_id":"CORiosRevA","site_address":"Blanca, CO 81123, USA","module_width":41.1,"module_length":75.08,"number_rows":"4","number_columns":6,"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":37,"ground_clearance":5,"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,"wind_speed_override":105,"snow_load_override":35,"direct_snow_load":false}

Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

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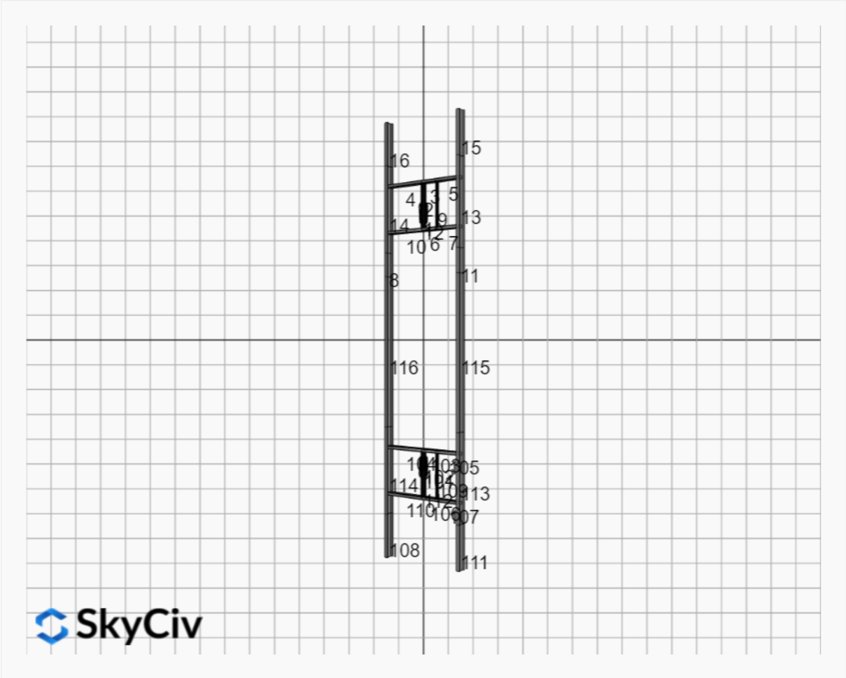
5.47
SkyCiv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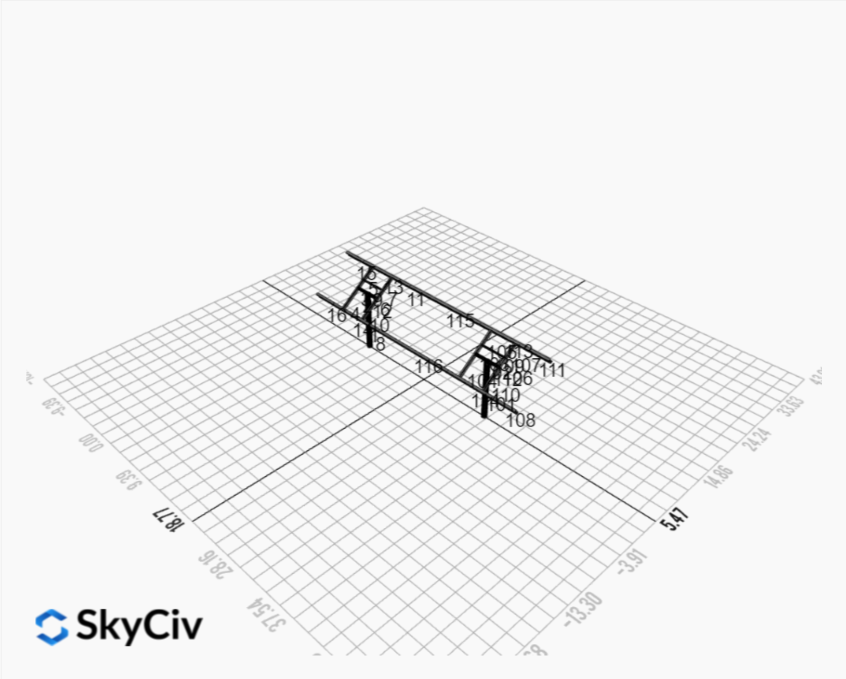
SkyCiv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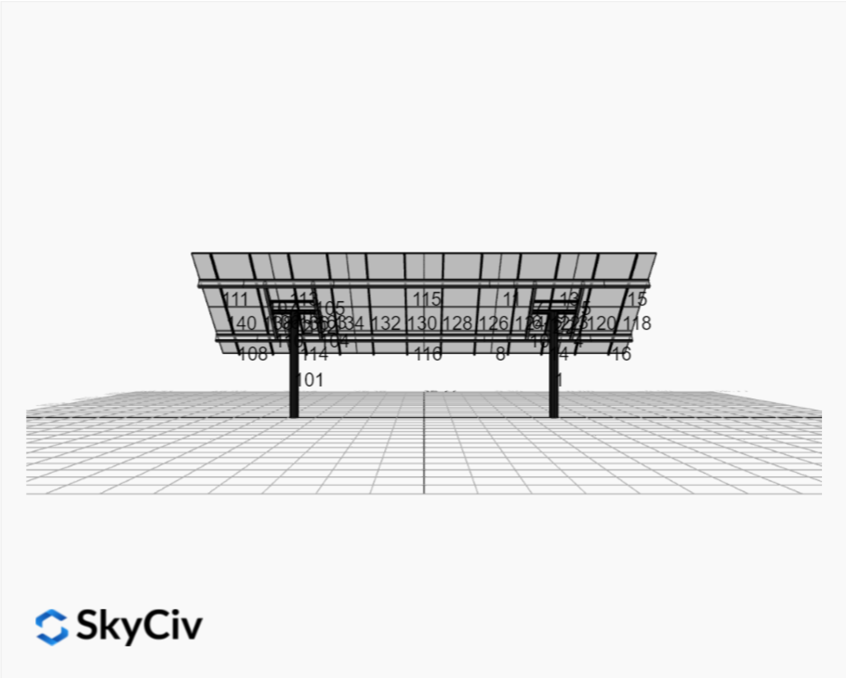
116
111
108
18.77
5.47
0.00
9.39
28.16
37.54
14.86
24.24
33.63
-3.91
-13.30
SkyCiv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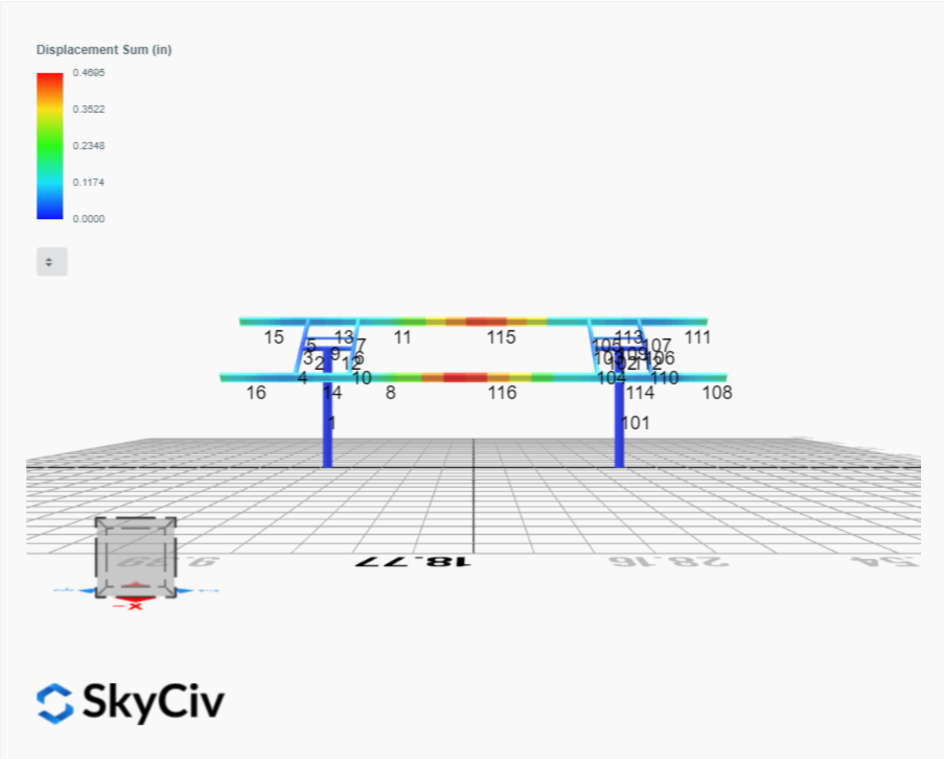
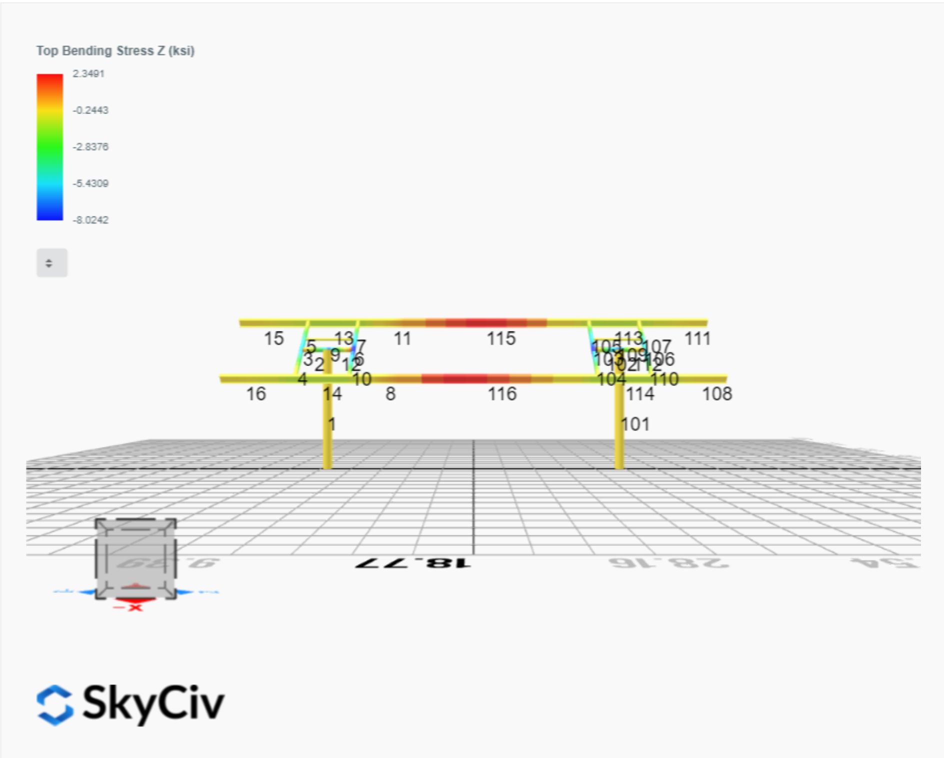
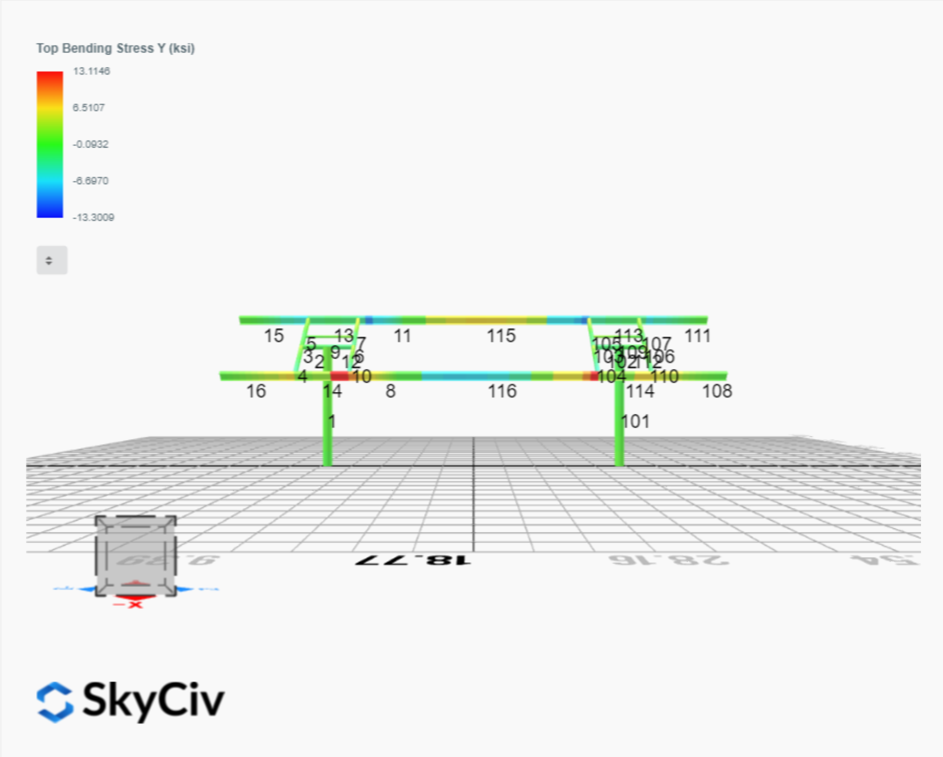
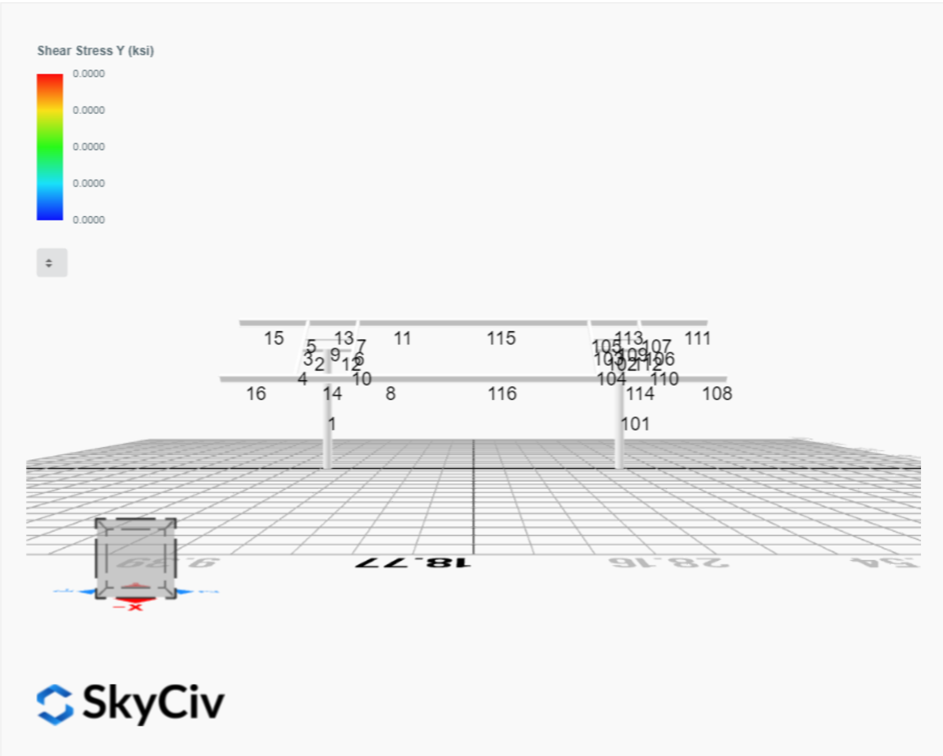
115
15
118
132
130
128
126
120
108
114
16
101
SkyCiv

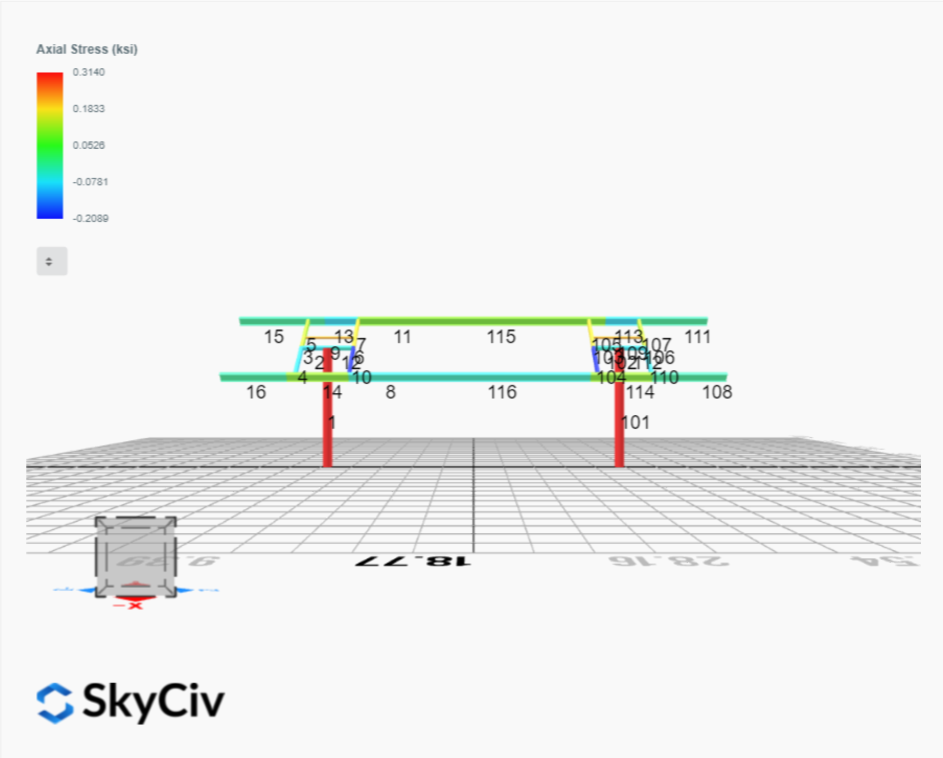
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
13.1146
6.5107
-0.0932
-6.6970
-13.3009


Shear Stress Y (ksi)
0.0000
0.0000
0.0000
0.0000
0.0000


Axial Stress (ksi)
0.3140
0.1833
0.0526
-0.0781
-0.2089
15
13
11
115
111
16
14
8
116
114
108
1
101
18.77
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	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 2. D + L	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 3. D + (S or Lr or R)	0.0000	4.7239	0.2327	0.5528	-0.2627	0.0339
ULS: 3. D + (S or Lr or R)	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.0646	0.1965	0.4669	-0.2218	0.0317
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 5b. D + 0.7E	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.0646	0.1965	0.4669	-0.2218	0.0317
ULS: 8. 0.6D + 0.7E	0.0000	1.2520	0.0529	0.1254	-0.0596	0.0150
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6725	5.6331	0.3311	0.7438	-0.8078	25.1893
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.6725	5.6331	0.3311	0.7438	-0.8078	25.1893
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2409	-0.8871	-0.1143	-0.2360	0.4926	-20.3361
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8674	-0.3915	-0.0819	-0.1646	0.3975	-23.9201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0044	6.7245	0.3788	0.8679	-0.7532	18.9050
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0044	6.7245	0.3788	0.8679	-0.7532	18.9050
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6807	1.8343	0.0447	0.1331	0.2221	-15.2391
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4006	2.2060	0.0691	0.1866	0.1508	-17.9271
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0044	4.7465	0.2704	0.6101	-0.6307	18.8982
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0044	4.7465	0.2704	0.6101	-0.6307	18.8982
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6807	-0.1437	-0.0637	-0.1247	0.3446	-15.2458
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4006	0.2280	-0.0394	-0.0712	0.2733	-17.9338
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6725	4.7985	0.2959	0.6601	-0.7681	25.1793
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.6725	4.7985	0.2959	0.6601	-0.7681	25.1793
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2409	-1.7218	-0.1496	-0.3196	0.5323	-20.3461
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8674	-1.2262	-0.1171	-0.2482	0.4373	-23.9301

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	9.7334
Shear X	-4.4541
Shear Z	0.5841
Moment X	1.3167
Moment Y (Twist)	1.3814
Moment Z	42.3420

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	6.7245
Shear X	-2.6725
Shear Z	0.3788
Moment X	0.8679
Moment Y (Twist)	0.8078
Moment Z	25.1893

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	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 2. D + L	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 3. D + (S or Lr or R)	-0.0000	4.7239	-0.2327	-0.5530	0.2629	0.0341
ULS: 3. D + (S or Lr or R)	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.0646	-0.1965	-0.4670	0.2220	0.0318
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 5b. D + 0.7E	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.0646	-0.1965	-0.4670	0.2220	0.0318
ULS: 8. 0.6D + 0.7E	-0.0000	1.2520	-0.0529	-0.1254	0.0597	0.0150
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6725	5.6331	-0.3311	-0.7438	0.8079	25.1894
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.6725	5.6331	-0.3311	-0.7438	0.8079	25.1894
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2409	-0.8871	0.1143	0.2359	-0.4925	-20.3361
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8674	-0.3915	0.0819	0.1646	-0.3975	-23.9200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0044	6.7244	-0.3788	-0.8681	0.7533	18.9051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0044	6.7244	-0.3788	-0.8681	0.7533	18.9051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6807	1.8342	-0.0447	-0.1333	-0.2220	-15.2390
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4006	2.2060	-0.0691	-0.1868	-0.1507	-17.9270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0044	4.7465	-0.2704	-0.6101	0.6308	18.8983
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0044	4.7465	-0.2704	-0.6101	0.6308	18.8983
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6807	-0.1437	0.0637	0.1247	-0.3445	-15.2458
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4006	0.2280	0.0394	0.0712	-0.2732	-17.9338
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6725	4.7985	-0.2959	-0.6602	0.7681	25.1793
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.6725	4.7985	-0.2959	-0.6602	0.7681	25.1793
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2409	-1.7218	0.1496	0.3196	-0.5323	-20.3461
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8674	-1.2262	0.1171	0.2482	-0.4372	-23.9300

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	9.7334
Shear X	-4.4542
Shear Z	-0.5841
Moment X	-1.3173
Moment Y (Twist)	1.3823
Moment Z	42.3432

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	6.7244
Shear X	-2.6725
Shear Z	-0.3788
Moment X	-0.8681
Moment Y (Twist)	0.8079
Moment Z	25.1894

Project Details

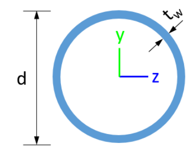
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	CORiosRevA
Unit System:	imperial

Design Input Information

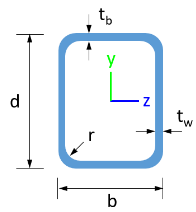
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

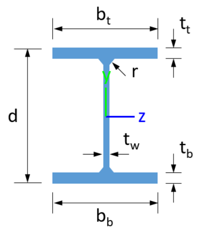
Section Dimensions



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

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	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	19.26	19.26	9.17	-	300	200	1
2	5	4.20	4.20	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.71,1.67,1.67,1.66,1.50,1.67,1.67,1.69,1.68,1.67,1.67,1.64,1.72,1.67,1.67,1.66,1.62	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.20,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	19	1.33	1.33	2.05	1.65,1.65,1.65,1.65,1.65,1.65,1.63,1.63,1.60,2.13,1.62,1.62,1.61,1.37,1.64,1.64,1.71,1.69,1.63,1.63,1.60,2.03,1.62,1.62,1.62,1.46	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.68,1.67,1.67,1.65,1.73,1.67,1.67,1.66,1.65	300	200	1
11	19	1.33	1.33	2.05	1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.64,1.62,1.65,1.64,1.64,1.63,1.66,1.65,1.65,1.73,1.73,1.65,1.65,1.61,1.65,1.64,1.64,1.63,1.66	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.09,1.10,1.14,1.09,1.09,1.09,1.13,1.09,1.09,1.07,1.12,1.09,1.09,1.10,1.14,1.09,1.09,1.09,1.13	300	200	1
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,2.27,1.09,1.09,1.09,1.22,1.08,1.08,1.07,1.11,1.08,1.08,1.10,1.50,1.09,1.09,1.09,1.15	300	200	1
15	19	7.88	7.88	3.75	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	19	7.88	7.88	3.75	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	9	19.26	19.26	9.17	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.20,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.68,1.67,1.67,1.65,1.73,1.67,1.67,1.66,1.65	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
106	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1

						0	0	
108	19	7.88	7.88	3.75	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	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.71,1.67,1.67,1.66,1.50,1.67,1.67,1.69,1.68,1.67,1.67,1.64,1.72,1.67,1.67,1.66,1.62	300	200	1
111	19	7.88	7.88	3.75	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	5	4.20	4.20	2.00	-	300	200	1
113	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.09,1.10,1.14,1.09,1.09,1.09,1.13,1.09,1.09,1.07,1.12,1.09,1.09,1.10,1.14,1.09,1.09,1.09,1.13	300	200	1
114	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,2.28,1.09,1.09,1.09,1.22,1.08,1.08,1.07,1.11,1.08,1.08,1.10,1.50,1.09,1.09,1.09,1.15	300	200	1
115	19	8.42	8.42	12.95	1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11	300	200	1
116	19	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.15,1.11,1.11,1.11,1.09,1.11,1.11,1.12,1.12,1.11,1.11,1.11,1.13,1.11,1.11,1.11,1.10	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	377.97	240.36	83.29	83.29	113.39	113.39
2	198.33	182.14	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	126.01	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	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.52	6.12	40.24	43.62
14	133.20	104.94	24.52	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	377.97	240.36	83.29	83.29	113.39	113.39
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	52.83	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	52.83	32.87	6.12	40.24	43.62
112	198.33	182.14	21.95	21.95	59.50	59.50
113	133.20	104.94	24.52	6.12	40.24	43.62
114	133.20	104.94	24.52	6.12	40.24	43.62

115	133.20	46.28	12.02	6.12	40.24	43.62
116	133.20	46.28	11.81	6.12	40.24	43.62

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	0.040	0.508	0.048	0.039	0.005	0.544	#13	0.393	Not Required	Pass
2	0.004	0.271	0.170	0.065	0.034	0.443	#13	0.114	Not Required	Pass
3	0.007	0.521	0.035	0.051	0.009	0.529	#13	0.045	Not Required	Pass
4	0.006	0.501	0.126	0.050	0.029	0.577	#13	0.080	Not Required	Pass
5	0.006	0.324	0.105	0.052	0.027	0.333	#13	0.074	Not Required	Pass
6	0.010	0.700	0.087	0.072	0.017	0.759	#13	0.045	Not Required	Pass
7	0.010	0.433	0.201	0.070	0.051	0.467	#13	0.074	Not Required	Pass
8	0.003	0.103	0.205	0.049	0.017	0.210	#24	0.095	Not Required	Pass
9	0.014	0.064	0.083	0.002	0.004	0.144	#13	0.204	Not Required	Pass
10	0.012	0.677	0.181	0.068	0.037	0.705	#13	0.080	Not Required	Pass
11	0.005	0.104	0.214	0.051	0.017	0.222	#21	0.095	Not Required	Pass
12	0.003	0.455	0.233	0.094	0.042	0.688	#13	0.053	Not Required	Pass
13	0.006	0.194	0.452	0.062	0.021	0.598	#21	0.286	Not Required	Pass
14	0.006	0.190	0.442	0.060	0.021	0.582	#21	0.190	Not Required	Pass
15	0.000	0.058	0.113	0.025	0.008	0.163	#21	Not Required	Not Required	Pass
16	0.000	0.056	0.113	0.024	0.008	0.162	#21	Not Required	Not Required	Pass
101	0.040	0.508	0.048	0.039	0.005	0.544	#13	0.393	Not Required	Pass
102	0.003	0.455	0.233	0.094	0.042	0.688	#13	0.053	Not Required	Pass
103	0.010	0.700	0.087	0.072	0.017	0.759	#13	0.045	Not Required	Pass
104	0.012	0.677	0.181	0.068	0.037	0.705	#13	0.080	Not Required	Pass
105	0.010	0.433	0.201	0.070	0.051	0.467	#13	0.074	Not Required	Pass
106	0.007	0.521	0.035	0.051	0.009	0.529	#13	0.045	Not Required	Pass
107	0.006	0.324	0.105	0.052	0.027	0.333	#13	0.074	Not Required	Pass
108	0.000	0.056	0.113	0.024	0.008	0.162	#21	Not Required	Not Required	Pass
109	0.014	0.064	0.083	0.002	0.004	0.144	#13	0.204	Not Required	Pass
110	0.006	0.501	0.126	0.050	0.029	0.577	#13	0.080	Not Required	Pass
111	0.000	0.058	0.113	0.025	0.008	0.163	#21	Not Required	Not Required	Pass
112	0.004	0.271	0.170	0.065	0.034	0.443	#13	0.114	Not Required	Pass
113	0.006	0.194	0.451	0.062	0.021	0.598	#21	0.190	Not Required	Pass
114	0.006	0.190	0.442	0.060	0.021	0.582	#21	0.286	Not Required	Pass
115	0.013	0.611	0.245	0.051	0.017	0.776	#21	0.601	Not Required	Pass
116	0.005	0.601	0.245	0.049	0.017	0.764	#21	0.601	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)

L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

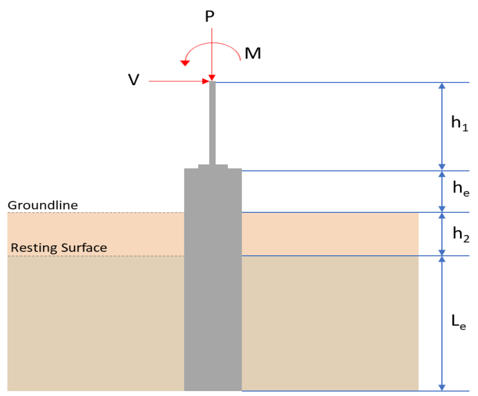
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.724	9.733
V_x (kip)	-2.672	-4.454
V_z (kip)	0.379	0.584
M_x (kipft)	0.868	1.317
M_z (kipft)	25.189	42.342

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.672\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42548\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.189\ \mathrm{kipft}) + ((-2.672\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.011\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 5.6192\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.379\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.06035\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.868\ \mathrm{kipft}) + ((0.379\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.13822\ \mathrm{kipft/ft}$$

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:
$L_{e,z} = 2.7614\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6192\ \mathrm{ft}), (2.7614\ \mathrm{ft})]$$

$$L_{e,req} = 5.619\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.25\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.619\ \mathrm{ft})}{(6.25\ \mathrm{ft})}$$

$$Ratio = 0.89904$$

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

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

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

$$q = \frac{(6.724\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.42025\ \mathrm{kip/ft}^2$$

$$q = 0.42025\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.42025\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.21013$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.42548\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.011\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.011\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.42548\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (4.011\ \mathrm{kipft/ft})) + (4 \times (-0.42548\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3263\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.011\ \mathrm{kipft/ft})) + (3 \times (-0.42548\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (4.011\ \mathrm{kipft/ft})) + (2 \times (-0.42548\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.18605\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.011\ \mathrm{kipft/ft})) + ((-0.42548\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.82372\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.3263\ \mathrm{ft})}{2}$$

$$p_a = 0.32447\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.18605\ \mathrm{kip/ft}^2\right)}{\left(0.32447\ \mathrm{kip/ft}^2\right)}$$

$$Ratio = 0.57339$$

Status: **PASS**
Ratio: **0.570**

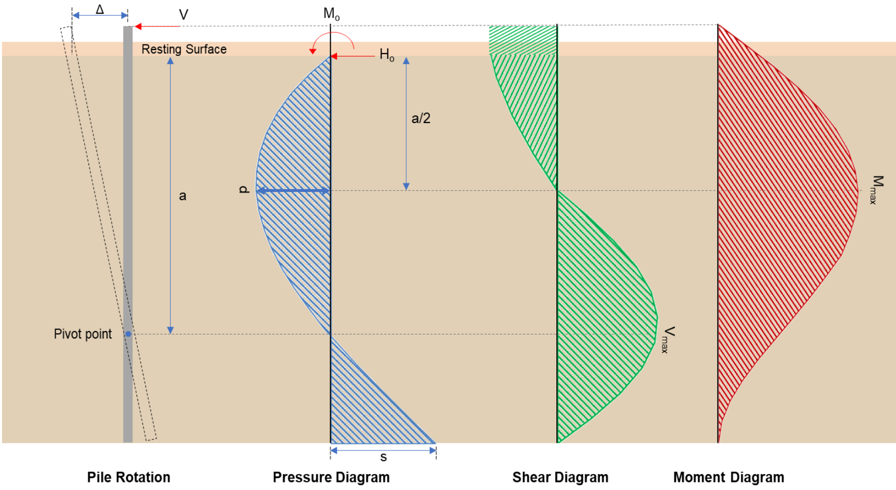
p_s - Allowable lateral soil pressure at depth L_e,

	$$p_s = R\,L_e$$ $$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$ $$p_s = 0.9375\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$ $$Ratio = \frac{\left(0.82372\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$ $$Ratio = 0.87863$$	Status: **PASS** Ratio: **0.880**	
	Considering z-direction: $H_o = 0.06035\ \mathrm{kip/ft}$ - Lateral force per length of pile, $M_o = 0.13822\ \mathrm{kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $$a = \frac{\left(4\,M_o\,L_e\right) + \left(3\,H_o\,L_e^2\right)}{\left(6\,M_o\right) + \left(4\,H_o\,L_e\right)}$$ $$a = \frac{\left(4 \times (0.13822\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})\right) + \left(3 \times (0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{\left(6 \times (0.13822\ \mathrm{kipft/ft})\right) + \left(4 \times (0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})\right)}$$ $$a = 4.5028\ \mathrm{ft}$$ p - Earth pressure against the pile at distance a/2 from resting surface, $$p = \frac{0.75\,\left[\left(4\,M_o\right) + \left(3\,H_o\,L_e\right)\right]^2}{L_e^2\,\left[\left(3\,M_o\right) + \left(2\,H_o\,L_e\right)\right]}$$ $$p = \frac{0.75 \times \left[\left(4 \times (0.13822\ \mathrm{kipft/ft})\right) + \left(3 \times (0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})\right)\right]^2}{(6.25\ \mathrm{ft})^2 \times \left[\left(3 \times (0.13822\ \mathrm{kipft/ft})\right) + \left(2 \times (0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})\right)\right]}$$ $$p = 0.0466\ \mathrm{kip/ft^2}$$ s - Earth pressure against the pile at distance L_e, $$s = \frac{6\,\left[\left(2\,M_o\right) + \left(H_o\,L_e\right)\right]}{L_e^2}$$ $$s = \frac{6 \times \left[\left(2 \times (0.13822\ \mathrm{kipft/ft})\right) + \left((0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})\right)\right]}{(6.25\ \mathrm{ft})^2}$$ $$s = 0.1004\ \mathrm{kip/ft^2}$$ **Check lateral soil pressure capacity:** p_a - Allowable lateral soil pressure at depth a/2, $$p_a = R\,\frac{a}{2}$$ $$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.5028\ \mathrm{ft})}{2}$$ $$p_a = 0.33771\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(0.0466\ \mathrm{kip/ft^2}\right)}{\left(0.33771\ \mathrm{kip/ft^2}\right)}$$ $$Ratio = 0.13799$$ p_s - Allowable lateral soil pressure at depth L_e, $$p_s = R\,L_e$$ $$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$ $$p_s = 0.9375\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$	Status: **PASS** Ratio: **0.140**	

$$Ratio = \frac{\left(0.1004\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.10709$$

Status: **PASS**
Ratio: **0.110**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.454\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.70924\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.342\ \mathrm{kipft}) + ((-4.454\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.7424\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7424\ \mathrm{kipft/ft})}{(-0.70924\ \mathrm{kip/ft})}$$

$$E = 9.5065\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7424\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.70924\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.7424\ \mathrm{kipft/ft})) + (4 \times (-0.70924\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3254\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.70924\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.5065\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.5065\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.5062\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.70924\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.25\ \mathrm{ft})) \times \left[\left(\frac{(9.5065\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + \frac{(4.3254\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (9.5065\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (9.5065\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 28.1\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.584\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.092994\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.317\ \mathrm{kipft}) + ((0.584\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.20971\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20971\ \mathrm{kipft/ft})}{(0.092994\ \mathrm{kip/ft})}$$

$$E = 2.2551\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20971\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.092994\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.20971\ \mathrm{kipft/ft})) + (4 \times (0.092994\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.5046\ \mathrm{ft}$$

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

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

$$V_{max} = ((0.092994\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2551\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.5046\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.2551\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.5046\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.48658\ \mathrm{kip}$$

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

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

$$M_{max} = ((0.092994\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.25\ \mathrm{ft})) \times \left[\left(\frac{(2.2551\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + \frac{(4.5046\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (2.2551\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.5046\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.2551\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.5046\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^4\right]\right]$$

	$$M_{max} = 1.3141 \text{ kipft}$$		

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.733 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.273 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.273 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.733\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0036383$$

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 $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.733\ \text{kip} \rightarrow 9733\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(9733\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.78\ \text{kip}$$

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

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.78\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.78\ \text{kip}$$

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

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

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

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

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

A_v - Ties rebar area,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$

$$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.78 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.94 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.5062 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.5062 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.085687$$

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

Considering z-direction:

$V_{max} = 0.48658 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.48658 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.004386$$

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:
$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 28.1 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(28.1 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11258$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$M_{max} = 1.3141 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(1.3141 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0052647$$

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

REFERENCES	CALCULATIONS	RESULTS

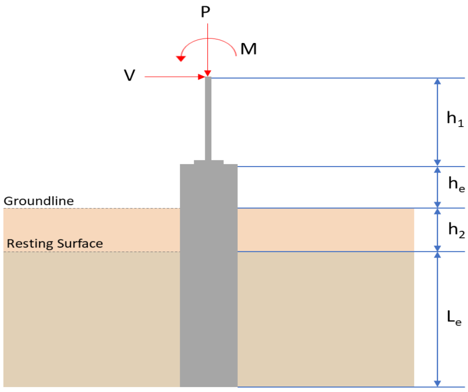
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.724	9.733
V_x (kip)	-2.672	-4.454
V_z (kip)	-0.379	-0.584
M_x (kipft)	-0.868	-1.317
M_z (kipft)	25.189	42.343

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.672\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42548\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.189\ \text{kipft}) + ((-2.672\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.011\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.6192\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.379\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.06035\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.868\ \text{kipft}) + ((-0.379\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.13822\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.6991\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6192\ \text{ft}), (1.6991\ \text{ft})]$$

$$L_{e,req} = 5.619\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.619\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.89904$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.724\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.42025\ \text{kip/ft}^2$$

$q = 0.42025 \text{ kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.42025 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.21013$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.42548 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.011 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.011 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.42548 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (4.011 \text{ kipft/ft})) + (4 \times (-0.42548 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3263 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.011 \text{ kipft/ft})) + (3 \times (-0.42548 \text{ kip/ft}) \times (6.25 \text{ ft}))]^{\ 2}}{(6.25 \text{ ft})^2 \times [(3 \times (4.011 \text{ kipft/ft})) + (2 \times (-0.42548 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.18605 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.011 \text{ kipft/ft})) + ((-0.42548 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.82372 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3263 \text{ ft})}{2}$$

$$p_a = 0.32447 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.18605 \text{ kip/ft}^2\right)}{\left(0.32447 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.57339$$

Status: **PASS**
Ratio: **0.570**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.82372\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.87863$$

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

Considering z-direction:

$H_o = -0.06035\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.13822\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13822\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.13822\ \mathrm{kipft/ft})) + (4 \times (-0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.5028\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.13822\ \mathrm{kipft/ft})) + (3 \times (-0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.13822\ \mathrm{kipft/ft})) + (2 \times (-0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.018927\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.13822\ \mathrm{kipft/ft})) + ((-0.06035\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = -0.015476\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.5028\ \mathrm{ft})}{2}$$

$$p_a = 0.33771\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.018927\ \mathrm{kip/ft^2}\right)}{\left(0.33771\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.056045$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

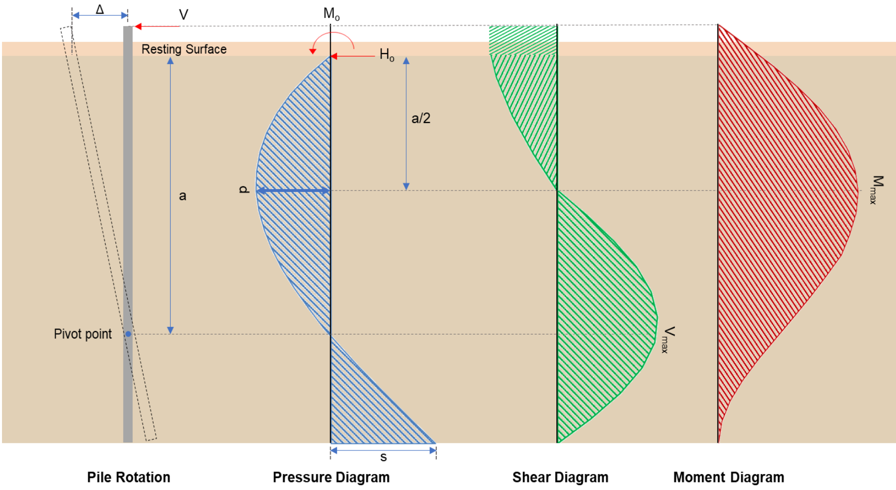
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(-0.015476\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = -0.016508$$

Status: **PASS**
Ratio: **-0.020**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.454\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.70924\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.343\ \mathrm{kipft}) + ((-4.454\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.7425\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7425\ \mathrm{kipft/ft})}{(-0.70924\ \mathrm{kip/ft})}$$

$$E = 9.5067\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7425\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.70924\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.7425\ \mathrm{kipft/ft})) + (4 \times (-0.70924\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3254\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.70924\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.5067\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.5067\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3254\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.5063\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.70924 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(9.5067 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3254 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.5067 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3254 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.5067 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3254 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.1 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.584 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.092994 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.317 \text{ kipft}) + ((-0.584 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20971 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20971 \text{ kipft/ft})}{(-0.092994 \text{ kip/ft})}$$

$$E = 2.2551 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20971 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.092994 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.20971 \text{ kipft/ft})) + (4 \times (-0.092994 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5046 \text{ ft}$$

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

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

$$V_{max} = ((-0.092994 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2551 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5046 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.2551 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5046 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.48658 \text{ kip}$$

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

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

$$M_{max} = ((-0.092994 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.2551 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5046 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.2551 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5046 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2551 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5046 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.3141 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.733 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.273 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.273 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.733\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0036383$$

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 $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.733\ \text{kip} \rightarrow 9733\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(9733\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.78\ \text{kip}$$

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

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.78\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.78\ \text{kip}$$

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{s,a} = 737.28\ \mathrm{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$ $$V_{s,b} = 50.894\ \mathrm{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((119.78\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 110.94\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 9.5063\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(9.5063\ \mathrm{kip})}{(110.94\ \mathrm{kip})}$$ $$Ratio = 0.085689$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $V_{max} = 0.48658\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.48658\ \mathrm{kip})}{(110.94\ \mathrm{kip})}$$ $$Ratio = 0.004386$$	Status: **PASS** Ratio: **0.000**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$	

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
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:** $M_{max} = 28.1 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(28.1 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.11258$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $M_{max} = 1.3141 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.3141 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0052647$$	Status: **PASS** Ratio: **0.010**