Your Project Calculations

Project Name: Telford Carport CU B

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Telford%20Carport%20CU%20B&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=6ln4IP1oxLhATFjog1a16fxsmiiggPbBER137P5y2aaYSLIfQQvRSJ1SrA1onA2V

Array Specification

Product:	Beam
Unique ID:	2P-17-8TOP-XD-57-L-5Hx6W-STRUTS-1271
Duty Classification:	XD
Module Width:	45.00 in
Module Length:	68.00in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	30
Front Edge Clearance:	7
Total Array Height at Tilt:	16.43 ft
Total Frame Length:	34.00 ft
Frame Weight:	2173 lbs
Array Dimensions N/S:	18.96 ft
Array Dimensions E/W:	34.50 ft
Rail Length:	227.50 in
Rail Spacing:	2.83 ft
Rail Check:	Not Checked

Support Specifications

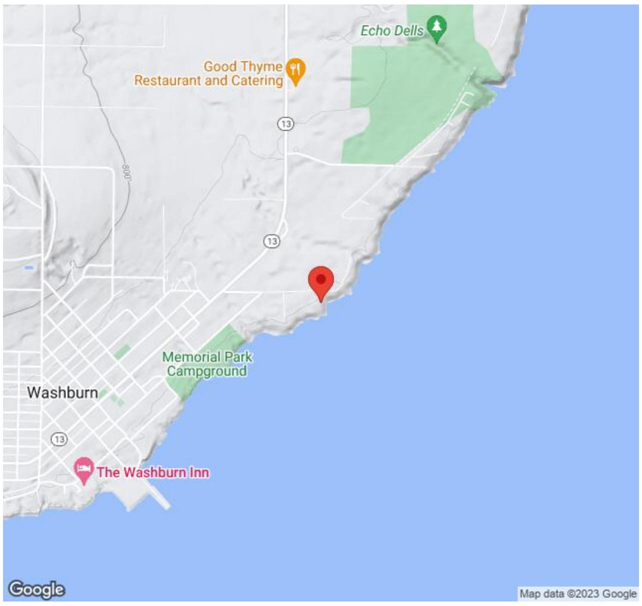
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.74 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft
Foundation Volume:	7.407 y^3
Foundation Result:	PASSED
Mount Twist:	0.811963 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	727 Superior Ave, Washburn, WI 54891, USA
Wind Speed:	98 mph
Snow Load:	60 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.026391 ksf



Design Disclaimer

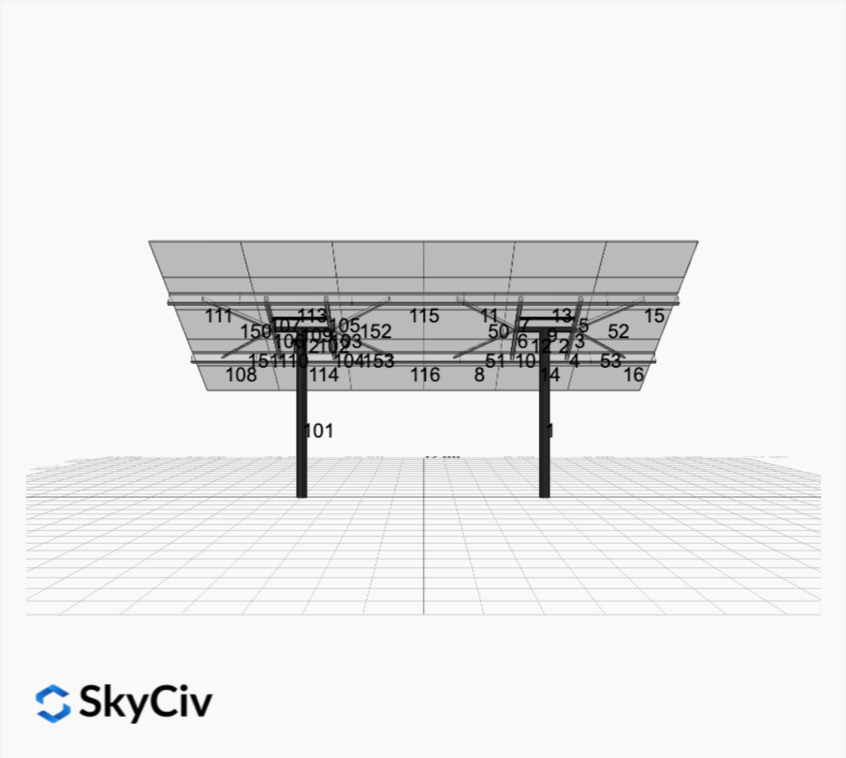
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

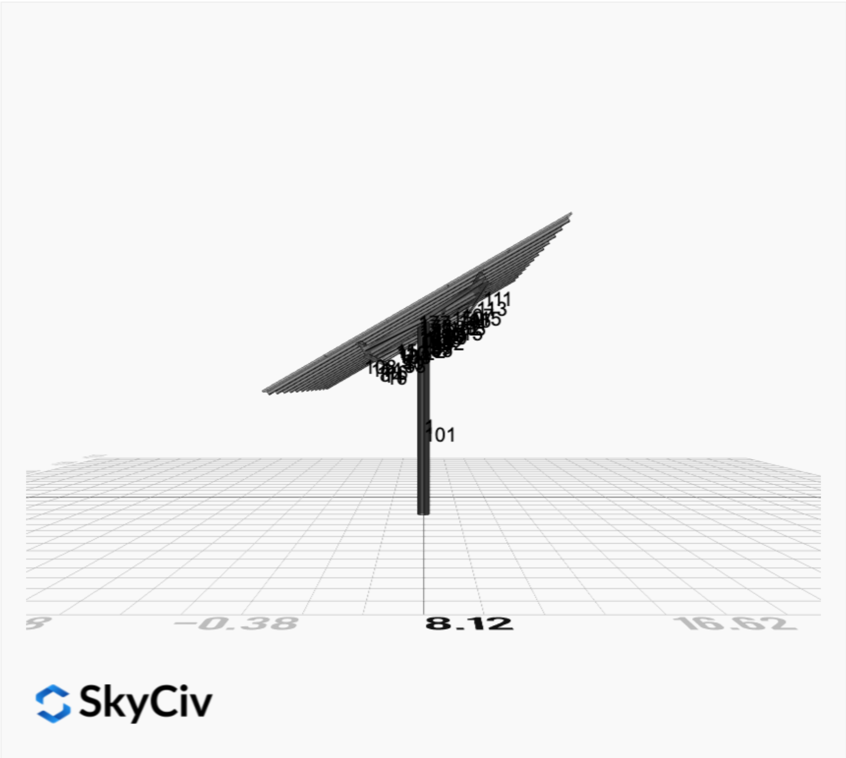
AutoDesigner Input

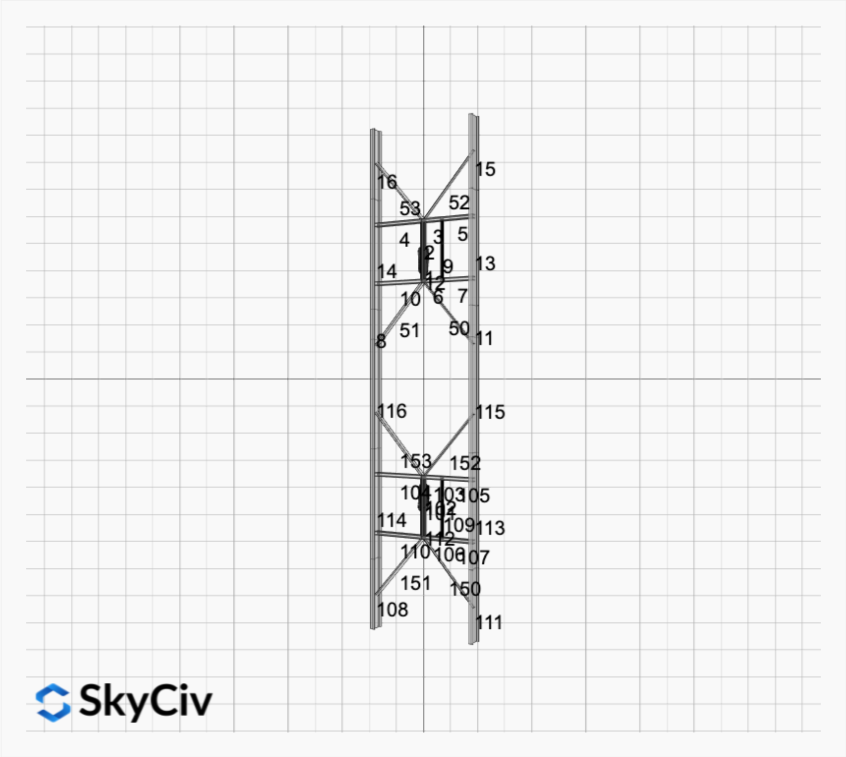
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":true,"product_type":"Beam","project_id":"Telford Carport CU B","site_address":"727 Superior Ave, Washburn, WI 54891, USA","module_width":45,"module_length":68,"number_rows":"5","number_columns":6,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":30,"ground_clearance":7,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}

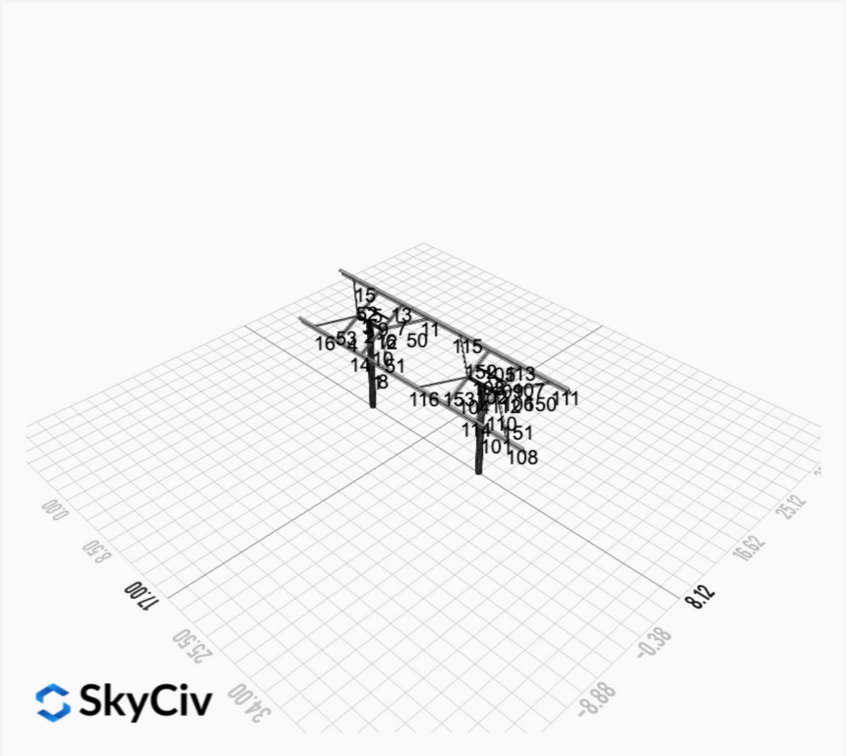
Design Notes:

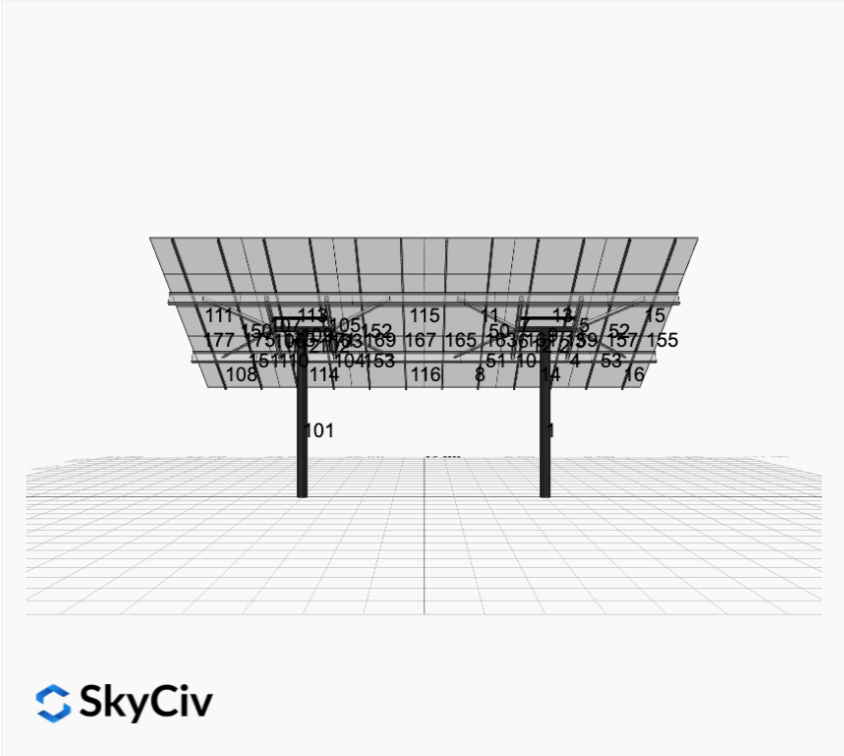
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Foundation Design and Sizing is approximate only


111
113
105
152
115
11
13
5
52
15
150
50
108
114
116
8
16
101
SkyCiv

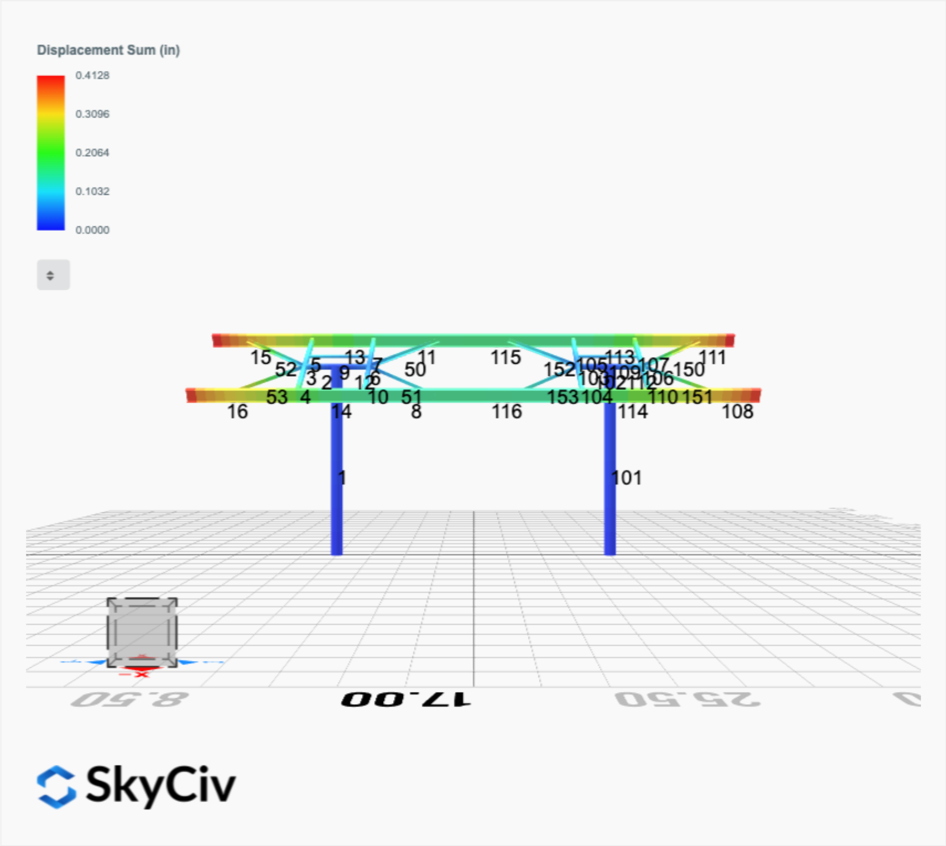

101
-0.38
8.12
16.62
SkyCiv

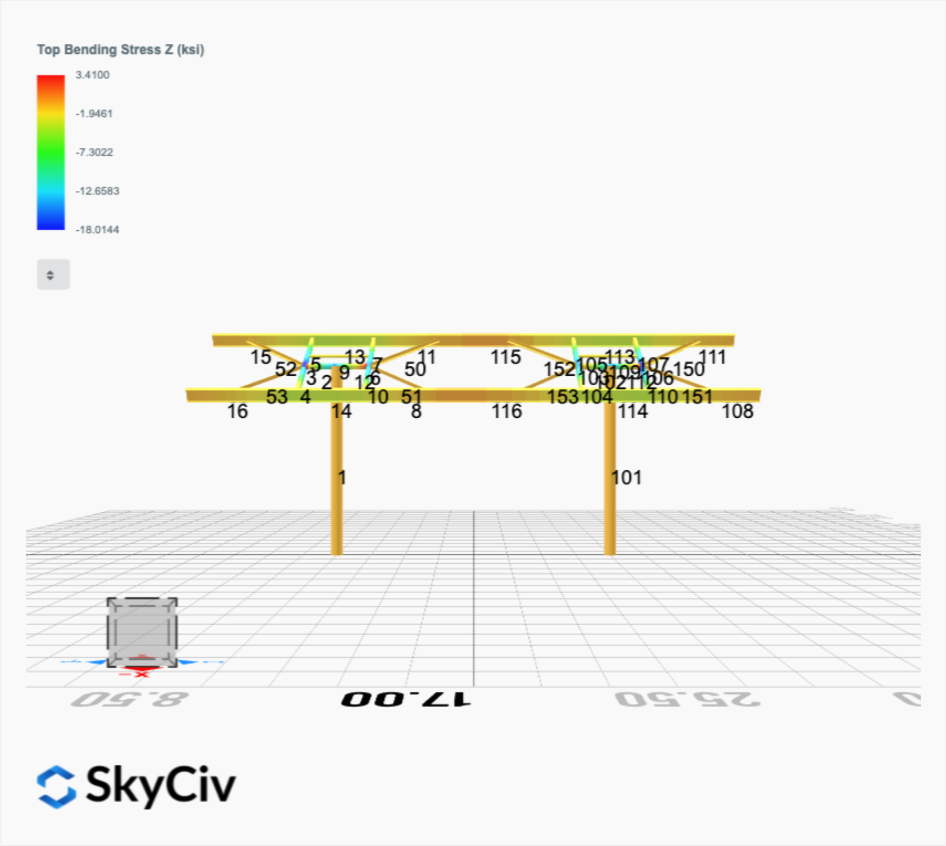


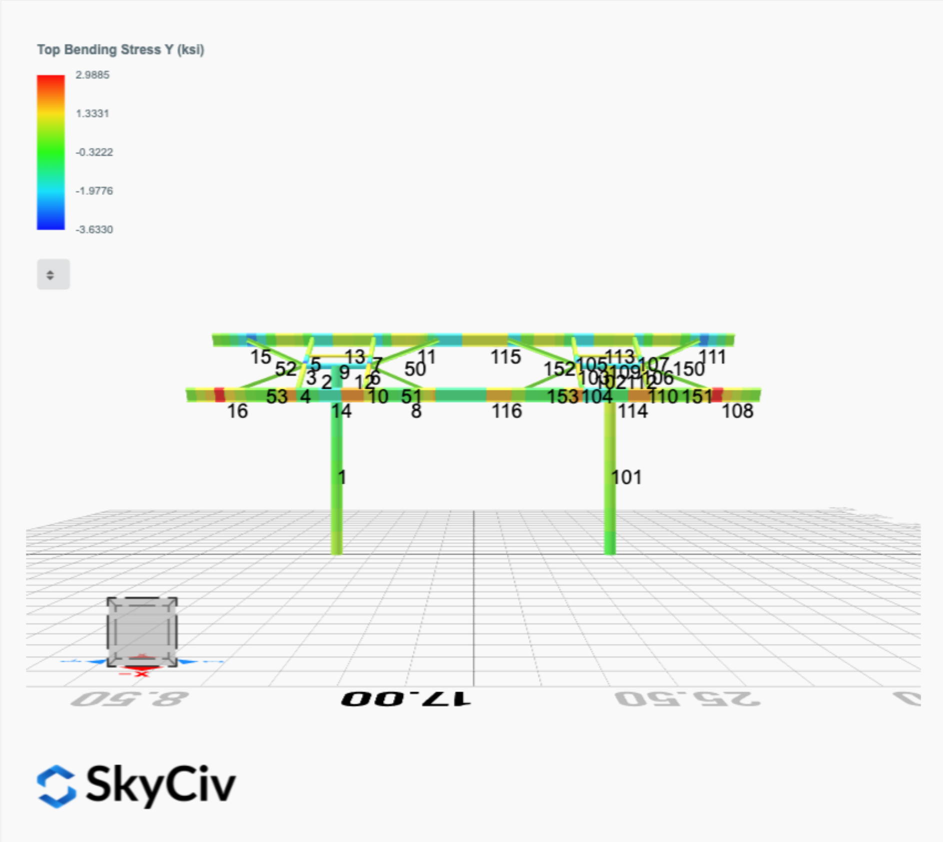


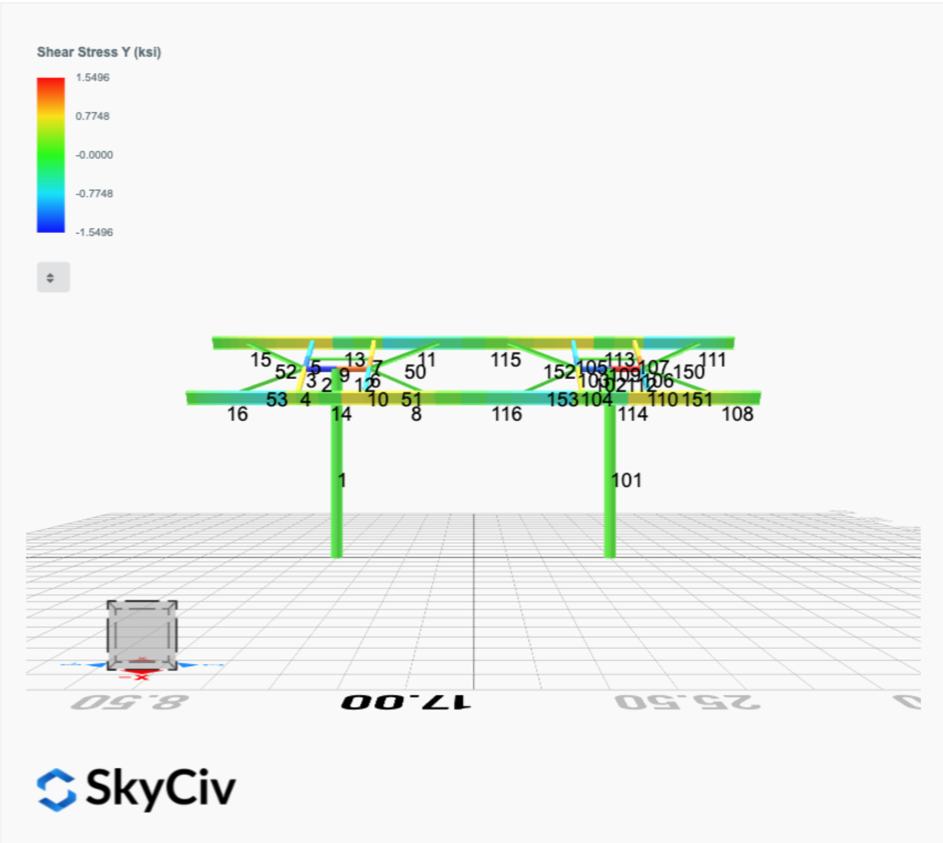

111
113
105
152
115
11
15
52
177
169
167
165
155
108
114
116
8
53
16
101

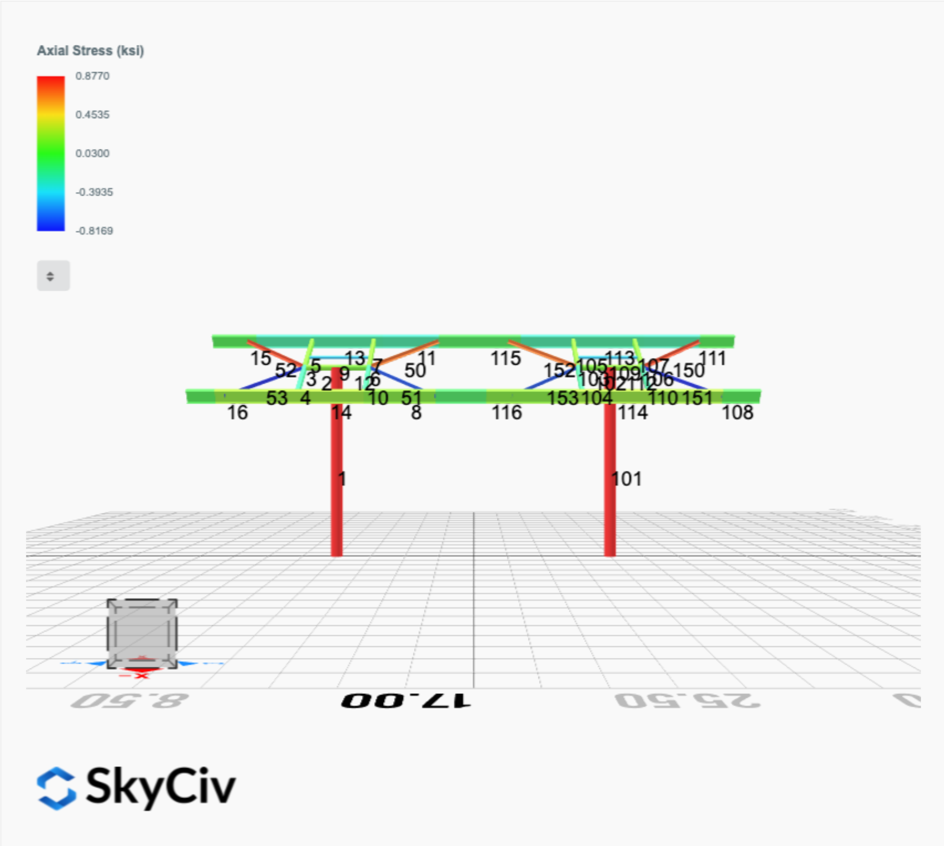
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
2.9885
1.3331
-0.3222
-1.9776
-3.6330
15
52
5
3
2
9
13
7
12
6
11
50
115
152
105
113
107
150
111
53
4
10
51
153
104
110
151
16
14
8
116
114
108
1
101
8.50
17.00
25.50
SkyCiv


Shear Stress Y (ksi)
1.5496
0.7748
-0.0000
-0.7748
-1.5496
15
52
5
3
2
9
13
7
12
6
11
50
115
152
105
113
107
150
111
53
4
10
51
153
104
110
151
16
14
8
116
114
108
1
101
8.50
17.00
25.50
SkyCiv


Axial Stress (ksi)
0.8770
0.4535
0.0300
-0.3935
-0.8169
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	2.5746	-0.0410	-0.1421	0.0877	0.0307
ULS: 2. D + L	0.0000	2.5746	-0.0410	-0.1421	0.0877	0.0307
ULS: 3. D + (S or Lr or R)	0.0000	9.9407	-0.1945	-0.6756	0.4159	0.0902
ULS: 3. D + (S or Lr or R)	0.0000	2.5746	-0.0410	-0.1421	0.0877	0.0307
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	8.0992	-0.1561	-0.5422	0.3339	0.0753
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.5746	-0.0410	-0.1421	0.0877	0.0307
ULS: 5b. D + 0.7E	0.0000	2.5746	-0.0410	-0.1421	0.0877	0.0307
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	8.0992	-0.1561	-0.5422	0.3339	0.0753
ULS: 8. 0.6D + 0.7E	0.0000	1.5448	-0.0246	-0.0852	0.0526	0.0184
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0553	6.1344	-0.1397	-0.4860	0.3009	24.6193
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0553	6.1344	-0.1397	-0.4860	0.3009	24.6193
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7616	-0.4767	0.0435	0.1511	-0.0951	-20.3256
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4680	0.0319	0.0299	0.1042	-0.0658	-24.1730
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5414	10.7691	-0.2301	-0.8001	0.4938	18.5168
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5414	10.7691	-0.2301	-0.8001	0.4938	18.5168
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3212	5.8108	-0.0928	-0.3223	0.1968	-15.1918
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1010	6.1922	-0.1029	-0.3575	0.2188	-18.0774
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5414	5.2445	-0.1150	-0.4000	0.2476	18.4721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5414	5.2445	-0.1150	-0.4000	0.2476	18.4721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3212	0.2862	0.0224	0.0778	-0.0494	-15.2365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1010	0.6676	0.0122	0.0426	-0.0274	-18.1221
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0553	5.1046	-0.1233	-0.4291	0.2658	24.6070
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.0553	5.1046	-0.1233	-0.4291	0.2658	24.6070
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7616	-1.5065	0.0599	0.2080	-0.1301	-20.3378
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4680	-0.9980	0.0463	0.1610	-0.1008	-24.1853

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	17.8418
Shear X	-3.4255
Shear Z	-0.3798
Moment X	-1.3242
Moment Y (Twist)	0.8120
Moment Z	42.0343

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	10.7691
Shear X	-2.0553
Shear Z	-0.2301
Moment X	-0.8001
Moment Y (Twist)	0.4938
Moment Z	24.6193

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	2.5746	0.0410	0.1421	-0.0877	0.0307
ULS: 2. D + L	-0.0000	2.5746	0.0410	0.1421	-0.0877	0.0307
ULS: 3. D + (S or Lr or R)	-0.0000	9.9407	0.1945	0.6756	-0.4159	0.0902
ULS: 3. D + (S or Lr or R)	-0.0000	2.5746	0.0410	0.1421	-0.0877	0.0307
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	8.0992	0.1561	0.5422	-0.3339	0.0753
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.5746	0.0410	0.1421	-0.0877	0.0307
ULS: 5b. D + 0.7E	-0.0000	2.5746	0.0410	0.1421	-0.0877	0.0307

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	8.0992	0.1561	0.5422	-0.3339	0.0753
ULS: 8. 0.6D + 0.7E	-0.0000	1.5448	0.0246	0.0853	-0.0526	0.0184
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0553	6.1344	0.1397	0.4860	-0.3009	24.6193
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0553	6.1344	0.1397	0.4860	-0.3009	24.6193
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7616	-0.4767	-0.0435	-0.1511	0.0951	-20.3255
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4680	0.0319	-0.0299	-0.1042	0.0658	-24.1730
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5414	10.7691	0.2301	0.8001	-0.4938	18.5168
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5414	10.7691	0.2301	0.8001	-0.4938	18.5168
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3212	5.8108	0.0928	0.3223	-0.1968	-15.1918
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1010	6.1922	0.1029	0.3575	-0.2188	-18.0774
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5414	5.2445	0.1150	0.4000	-0.2476	18.4721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5414	5.2445	0.1150	0.4000	-0.2476	18.4721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3212	0.2862	-0.0224	-0.0778	0.0494	-15.2365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1010	0.6676	-0.0122	-0.0426	0.0274	-18.1221
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0553	5.1046	0.1233	0.4291	-0.2658	24.6070
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.0553	5.1046	0.1233	0.4291	-0.2658	24.6070
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7616	-1.5065	-0.0599	-0.2080	0.1301	-20.3378
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4680	-0.9980	-0.0463	-0.1610	0.1008	-24.1853

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	17.8419
Shear X	-3.4254
Shear Z	0.3798
Moment X	1.3245
Moment Y (Twist)	0.8120
Moment Z	42.0343

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	10.7691
Shear X	-2.0553
Shear Z	0.2301
Moment X	0.8001
Moment Y (Twist)	0.4938
Moment Z	24.6193

Project Details

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

Design Input Information

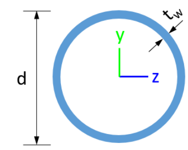
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

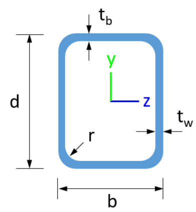
Design Materials

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

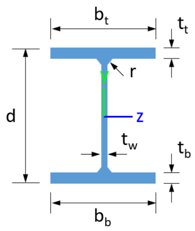
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



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



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	24.65	24.65	11.74	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.61,1.68,1.67,1.67,1.66,1.80,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.70,1.67,1.67,1.66,1.53	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.65,1.67,1.67,1.66,1.66	300	200	1
8	20	1.33	1.33	2.05	1.76,1.75,1.76,1.75,1.76,1.76,1.75,1.75,1.73,1.80,1.75,1.75,1.75,2.29,1.75,1.75,1.76,1.76,1.75,1.75,1.74,1.83,1.75,1.75,1.75,1.29	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.60,1.68,1.67,1.67,1.65,1.75,1.67,1.67,1.67,1.67,1.67,1.67,1.61,1.70,1.67,1.67,1.66,2.61	300	200	1
11	20	1.33	1.33	2.05	1.76,1.76,1.76,1.76,1.76,1.77,1.76,1.76,1.74,1.81,1.76,1.76,1.76,1.78,1.76,1.76,1.76,1.75,1.76,1.76,1.75,1.79,1.76,1.76,1.76,1.78	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.17,1.08,1.08,1.08,1.10,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.14,1.08,1.08,1.08,1.09	300	200	1
14	20	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.13,1.08,1.08,1.08,3.12,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.17,1.08,1.08,1.08,1.18	300	200	1
15	20	9.97	9.97	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.30,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.32,2.32,2.32,2.32,2.29,2.30,2.32,2.32,2.31,2.32	300	200	1
16	20	9.97	9.97	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.30,2.31,2.32,2.32,2.31,2.36,2.32,2.32,2.32,2.31,2.32,2.32,2.28,2.34,2.32,2.32,2.31,2.22	300	200	1
101	9	24.65	24.65	11.74	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
104	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.60,1.68,1.67,1.67,1.65,1.75,1.67,1.67,1.67,1.67,1.67,1.67,1.61,1.70,1.67,1.67,1.66,2.61	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	30	20	1

				[illegible]	[illegible]	0	0	
108	20	9.97	9.97	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.30,2.31,2.32,2.32,2.31,2.36,2.32,2.32,2.32,2.31,2.32,2.32,2.28,2.34,2.32,2.32,2.31,2.21	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.61,1.68,1.67,1.67,1.66,1.80,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.70,1.67,1.67,1.66,1.53	300	200	1
111	20	9.97	9.97	4.75	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.30,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.32,2.32,2.32,2.32,2.29,2.31,2.32,2.32,2.31,2.32	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1
113	20	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.17,1.08,1.08,1.08,1.10,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.14,1.08,1.08,1.08,1.09	300	200	1
114	20	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.13,1.08,1.08,1.08,3.11,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.17,1.08,1.08,1.08,1.18	300	200	1
115	20	4.84	4.84	7.45	2.35,2.35,2.35,2.36,2.35,2.35,2.36,2.36,2.40,2.26,2.35,2.35,2.36,2.30,2.36,2.36,2.36,2.37,2.35,2.35,2.36,2.28,2.35,2.35,2.36,2.31	300	200	1
116	20	4.84	4.84	7.45	2.37,2.37,2.37,2.37,2.37,2.37,2.37,2.37,2.42,2.26,2.37,2.37,2.37,1.30,2.37,2.37,2.36,2.35,2.37,2.37,2.39,2.23,2.37,2.37,2.37,2.01	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	180.07	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	33.01	6.46	56.26	44.91
14	159.30	116.35	33.01	6.46	56.26	44.91
15	159.30	34.37	46.90	6.46	56.26	44.91
16	159.30	34.37	46.90	6.46	56.26	44.91
101	377.97	180.07	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	34.37	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	34.37	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	33.01	6.46	56.26	44.91
114	159.30	116.35	33.01	6.46	56.26	44.91

115	159.30	104.63	46.90	6.46	56.26	44.91
116	159.30	104.63	39.96	6.46	56.26	44.91

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	0.099	0.505	0.037	0.030	0.003	0.552	#13	0.504	Not Required	Pass
2	0.006	0.592	0.147	0.123	0.025	0.721	#21	0.054	Not Required	Pass
3	0.005	0.791	0.136	0.080	0.053	0.929	#21	0.046	Not Required	Pass
4	0.006	0.783	0.039	0.078	0.005	0.815	#21	0.082	Not Required	Pass
5	0.005	0.490	0.017	0.078	0.004	0.508	#21	0.076	Not Required	Pass
6	0.006	0.686	0.125	0.068	0.051	0.810	#21	0.046	Not Required	Pass
7	0.006	0.426	0.024	0.068	0.005	0.453	#21	0.076	Not Required	Pass
8	0.006	0.078	0.025	0.039	0.007	0.094	#21	0.102	Not Required	Pass
9	0.022	0.089	0.055	0.002	0.001	0.155	#21	0.137	Not Required	Pass
10	0.006	0.678	0.078	0.068	0.014	0.758	#21	0.082	Not Required	Pass
11	0.007	0.078	0.025	0.039	0.008	0.105	#21	0.068	Not Required	Pass
12	0.005	0.477	0.124	0.108	0.021	0.575	#21	0.054	Not Required	Pass
13	0.007	0.311	0.051	0.054	0.010	0.354	#21	0.204	Not Required	Pass
14	0.011	0.314	0.067	0.053	0.012	0.386	#21	0.306	Not Required	Pass
15	0.007	0.112	0.081	0.039	0.011	0.130	#21	0.508	Not Required	Pass
16	0.031	0.111	0.081	0.039	0.011	0.131	#21	0.508	Not Required	Pass
101	0.099	0.505	0.037	0.030	0.003	0.552	#13	0.504	Not Required	Pass
102	0.005	0.477	0.124	0.108	0.021	0.575	#21	0.054	Not Required	Pass
103	0.006	0.686	0.125	0.068	0.051	0.810	#21	0.046	Not Required	Pass
104	0.006	0.678	0.078	0.068	0.014	0.758	#21	0.082	Not Required	Pass
105	0.006	0.426	0.024	0.068	0.005	0.453	#21	0.076	Not Required	Pass
106	0.005	0.791	0.136	0.080	0.053	0.929	#21	0.046	Not Required	Pass
107	0.005	0.490	0.017	0.078	0.004	0.508	#21	0.076	Not Required	Pass
108	0.031	0.111	0.081	0.039	0.011	0.131	#21	0.763	Not Required	Pass
109	0.022	0.089	0.055	0.002	0.001	0.155	#21	0.137	Not Required	Pass
110	0.006	0.783	0.039	0.078	0.005	0.815	#21	0.082	Not Required	Pass
111	0.007	0.112	0.081	0.039	0.011	0.130	#21	0.508	Not Required	Pass
112	0.006	0.592	0.147	0.123	0.025	0.721	#21	0.054	Not Required	Pass
113	0.007	0.311	0.051	0.054	0.010	0.354	#21	0.204	Not Required	Pass
114	0.011	0.314	0.067	0.053	0.012	0.386	#21	0.306	Not Required	Pass
115	0.007	0.078	0.048	0.039	0.010	0.105	#21	0.247	Not Required	Pass
116	0.008	0.078	0.052	0.039	0.010	0.095	#21	0.370	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

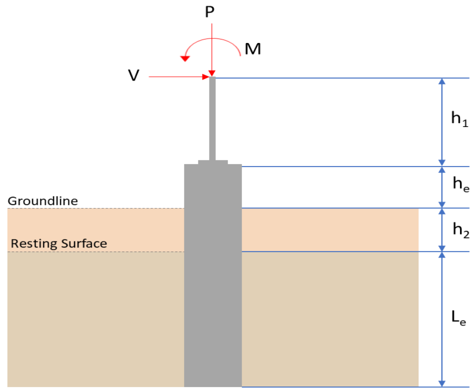
SkyCiv Foundation Design

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)	10.769	17.842
V_x (kip)	-2.055	-3.425
V_z (kip)	-0.230	-0.380
M_x (kipft)	-0.800	-1.324
M_z (kipft)	24.619	42.034

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,

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

$$H_o = \frac{(-2.055\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.32723\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.619\ \text{kipft}) + ((-2.055\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.9202\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8382\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.23\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.036624\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.8\ \text{kipft}) + ((-0.23\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.12739\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.8334\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8382\ \text{ft}), (1.8334\ \text{ft})]$$

$$L_{e,req} = 5.838\ \text{ft}$$

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

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

$$Ratio = \frac{(5.838\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.93408$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(10.769\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.67306\ \text{kip/ft}^2$$

$q = 0.67306\ \text{kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.67306\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.33653$$

Status: **PASS**
Ratio: **0.340**

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:

$H_o = -0.32723\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.9202\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9202\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (3.9202\ \text{kipft/ft})) + (4 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3011\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9202\ \text{kipft/ft})) + (3 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]^{\ 2}}{(6.25\ \text{ft})^2 \times [(3 \times (3.9202\ \text{kipft/ft})) + (2 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]}$$

$$p = 0.22807\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.9202\ \text{kipft/ft})) + ((-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]}{(6.25\ \text{ft})^2}$$

$$s = 0.89015\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.3011\ \text{ft})}{2}$$

$$p_a = 0.32258\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22807\ \text{kip/ft}^2\right)}{\left(0.32258\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.70702$$

Status: **PASS**
Ratio: **0.710**

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

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

$$Ratio = \frac{(0.89015\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.9495$$

Status: PASS
Ratio: 0.950

Considering z-direction:

$H_o = -0.036624\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.12739\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12739\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.036624\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.12739\ \mathrm{kipft/ft})) + (4 \times (-0.036624\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4505\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.12739\ \mathrm{kipft/ft})) + (3 \times (-0.036624\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.12739\ \mathrm{kipft/ft})) + (2 \times (-0.036624\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.0079661\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.12739\ \mathrm{kipft/ft})) + ((-0.036624\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.0039745\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4505\ \mathrm{ft})}{2}$$

$$p_a = 0.33379\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(-0.0079661\ \mathrm{kip/ft^2})}{(0.33379\ \mathrm{kip/ft^2})}$$

$$Ratio = -0.023866$$

Status: PASS
Ratio: -0.020

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

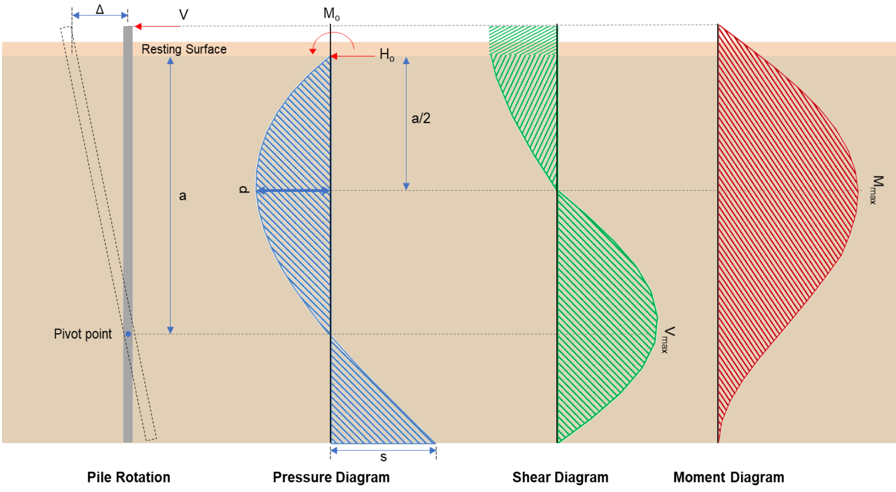
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0039745\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.0042395$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.425\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.54538\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.034\ \mathrm{kipft}) + ((-3.425\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.6933\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.6933\ \mathrm{kipft/ft})}{(-0.54538\ \mathrm{kip/ft})}$$

$$E = 12.273\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6933\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.54538\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.6933\ \mathrm{kipft/ft})) + (4 \times (-0.54538\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2987\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.54538\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.273\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.2987\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.273\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.2987\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.0201\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.54538 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(12.273 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2987 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.273 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2987 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.273 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2987 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.948 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.38 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.06051 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.324 \text{ kipft}) + ((-0.38 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21083 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21083 \text{ kipft/ft})}{(-0.06051 \text{ kip/ft})}$$

$$E = 3.4842 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21083 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.06051 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.21083 \text{ kipft/ft})) + (4 \times (-0.06051 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4503 \text{ ft}$$

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

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

$$V_{max} = ((-0.06051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4842 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4503 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.4842 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4503 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39976 \text{ kip}$$

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

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

$$M_{max} = ((-0.06051 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.4842 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4503 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.4842 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4503 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.4842 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4503 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.1141 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(17.842 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.003 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.003 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

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**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** 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 $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(17.842\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0066695$$	Status: **PASS** Ratio: **0.010**
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$ 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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $V_{c,max}$ - 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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 17.842\text{ kip} \rightarrow 17842\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(17842\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.86\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.86\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.86\text{ kip}$$	

	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) $$V_{s,b} = \frac{2\ A_v\ f_{ywk}\ d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times (0.11045\ \text{in}^2) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$ $$V_{s,b} = 50.894\ \text{kip}$$ 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 $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.86\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 111.64\ \text{kip}$$ **Considering x-direction:** $V_{max} = 9.0201\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(9.0201\ \text{kip})}{(111.64\ \text{kip})}$$ $$Ratio = 0.080794$$	Status: **PASS** Ratio: **0.080**
	Considering z-direction: $V_{max} = 0.39976\ \text{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.39976\ \text{kip})}{(111.64\ \text{kip})}$$ $$Ratio = 0.0035807$$	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:** $M_{max} = 26.948 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(26.948 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.10796$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $M_{max} = 1.1141 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.1141 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0044636$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

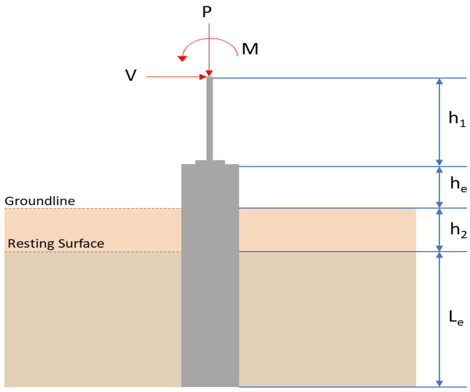
SkyCiv Foundation Design

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
$L = 6.25\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	10.769	17.842
V_x (kip)	-2.055	-3.425
V_z (kip)	0.230	0.380
M_x (kipft)	0.800	1.325
M_z (kipft)	24.619	42.034

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,

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

$$H_o = \frac{(-2.055\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.32723\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.619\ \text{kipft}) + ((-2.055\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.9202\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8382\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.23\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.036624\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.8\ \text{kipft}) + ((0.23\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.12739\ \text{kipft/ft}$$

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:
$L_{e,z} = 2.5036\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8382\ \text{ft}), (2.5036\ \text{ft})]$$

$$L_{e,req} = 5.838\ \text{ft}$$

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

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

$$Ratio = \frac{(5.838\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.93408$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(10.769\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.67306\ \text{kip/ft}^2$$

$q = 0.67306\ \text{kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.67306\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.33653$$

Status: **PASS**
Ratio: **0.340**

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:

$H_o = -0.32723\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.9202\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9202\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (3.9202\ \text{kipft/ft})) + (4 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3011\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9202\ \text{kipft/ft})) + (3 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]^{\,2}}{(6.25\ \text{ft})^2 \times [(3 \times (3.9202\ \text{kipft/ft})) + (2 \times (-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]}$$

$$p = 0.22807\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.9202\ \text{kipft/ft})) + ((-0.32723\ \text{kip/ft}) \times (6.25\ \text{ft}))]}{(6.25\ \text{ft})^2}$$

$$s = 0.89015\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.3011\ \text{ft})}{2}$$

$$p_a = 0.32258\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.22807\ \text{kip/ft}^2)}{(0.32258\ \text{kip/ft}^2)}$$

$$Ratio = 0.70702$$

Status: **PASS**
Ratio: **0.710**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.89015\ \text{kip/ft}^2\right)}{\left(0.9375\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.9495$$

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = 0.036624\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.12739\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12739\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (0.036624\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.12739\ \text{kipft/ft})) + (4 \times (0.036624\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.4505\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.12739\ \text{kipft/ft})) + (3 \times (0.036624\ \text{kip/ft}) \times (6.25\ \text{ft}))]^2}{(6.25\ \text{ft})^2 \times [(3 \times (0.12739\ \text{kipft/ft})) + (2 \times (0.036624\ \text{kip/ft}) \times (6.25\ \text{ft}))]}$$

$$p = 0.032711\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.12739\ \text{kipft/ft})) + ((0.036624\ \text{kip/ft}) \times (6.25\ \text{ft}))]}{(6.25\ \text{ft})^2}$$

$$s = 0.074293\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.4505\ \text{ft})}{2}$$

$$p_a = 0.33379\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.032711\ \text{kip/ft}^2\right)}{\left(0.33379\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.097997$$

Status: **PASS**
Ratio: **0.100**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

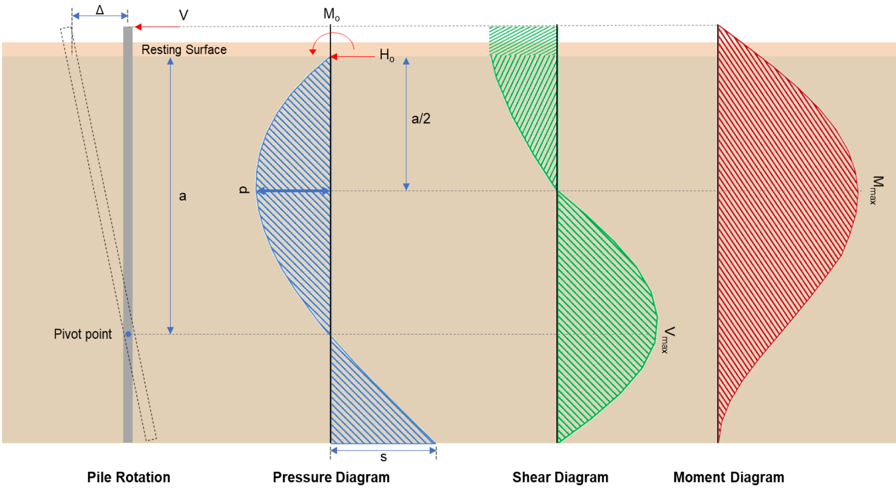
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.074293 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.079246$$

Status: **PASS**
Ratio: **0.080**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.425 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.54538 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.034 \text{ kipft}) + ((-3.425 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.6933 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.6933 \text{ kipft/ft})}{(-0.54538 \text{ kip/ft})}$$

$$E = 12.273 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6933 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.54538 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (6.6933 \text{ kipft/ft})) + (4 \times (-0.54538 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2987 \text{ ft}$$

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

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

$$V_{max} = ((-0.54538 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.273 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.2987 \text{ ft})}{(6.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.273 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.2987 \text{ ft})}{(6.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 9.0201 \text{ kip}$$

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

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

$$M_{max} = ((-0.54538\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(12.273\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.2987\ \text{ft})}{2 \times (6.25\ \text{ft})} \right) - \left[\left(\frac{4 \times (12.273\ \text{ft})}{(6.25\ \text{ft})} + 3 \right) \times \left(\frac{(4.2987\ \text{ft})}{2 \times (6.25\ \text{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.273\ \text{ft})}{(6.25\ \text{ft})} + 2 \right) \times \left(\frac{(4.2987\ \text{ft})}{2 \times (6.25\ \text{ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.948\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.38\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.06051\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.325\ \text{kipft}) + ((0.38\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.21099\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21099\ \text{kipft/ft})}{(0.06051\ \text{kip/ft})}$$

$$E = 3.4868\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21099\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (0.06051\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.21099\ \text{kipft/ft})) + (4 \times (0.06051\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.4502\ \text{ft}$$

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

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

$$V_{max} = ((0.06051\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4868\ \text{ft})}{(6.25\ \text{ft})} + 3 \right) \times \left(\frac{(4.4502\ \text{ft})}{(6.25\ \text{ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.4868\ \text{ft})}{(6.25\ \text{ft})} + 2 \right) \times \left(\frac{(4.4502\ \text{ft})}{(6.25\ \text{ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39994\ \text{kip}$$

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

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

$$M_{max} = ((0.06051\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(3.4868\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.4502\ \text{ft})}{2 \times (6.25\ \text{ft})} \right) - \left[\left(\frac{4 \times (3.4868\ \text{ft})}{(6.25\ \text{ft})} + 3 \right) \times \left(\frac{(4.4502\ \text{ft})}{2 \times (6.25\ \text{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.4868\ \text{ft})}{(6.25\ \text{ft})} + 2 \right) \times \left(\frac{(4.4502\ \text{ft})}{2 \times (6.25\ \text{ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.1147 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(17.842 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.003 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.003 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

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,

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

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

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

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,

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

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

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

Summary:

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

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

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \mathrm{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(17.842\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$ $$Ratio = 0.0066695$$	Status: **PASS** Ratio: **0.010**

	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\ \mathrm{in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\ \mathrm{in})$$ $$d = 38.4\ \mathrm{in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\ \mathrm{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,max} = 296.21\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 17.842\ \mathrm{kip} \rightarrow 17842\ \mathrm{lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(17842\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,a} = 120.86\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + (0.05 \times (2500\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,b} = 348.89\ \mathrm{kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\,[V_{c,max}, V_{c,a}, V_{c,b}]$$ $$V_c = Min\,[(296.21\ \mathrm{kip}), (120.86\ \mathrm{kip}), (348.89\ \mathrm{kip})]$$ $$V_c = 120.86\ \mathrm{kip}$$	

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

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

$$\phi V_n = (0.65) \times ((120.86 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.64 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.0201 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.0201 \text{ kip})}{(111.64 \text{ kip})}$$

$$Ratio = 0.080794$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$V_{max} = 0.39994 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39994 \text{ kip})}{(111.64 \text{ kip})}$$

$$Ratio = 0.0035823$$

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:** $M_{max} = 26.948\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(26.948\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.10796$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $M_{max} = 1.1147\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.1147\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.0044658$$	Status: **PASS** Ratio: **0.000**