Your Project Calculations

Project Name: MTSOLAR_K8I42DL04KIK

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MTSOLAR_K8I42DL04KIK&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=s12ld5N8Tr5eFnnxf6diWqxTTU7Fi436HVHQWIFHnqzbBqD28ptl83Sdltyac5Br

Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-HD-24-L-4Hx9W-I8E9
Duty Classification:	HD
Module Width:	44.65 in
Module Length:	67.88in
Number of Rows:	4
Number of Columns:	9
Total Number of Modules:	36
Desired Tilt Angle:	10
Front Edge Clearance:	8
Total Array Height at Tilt:	10.60 ft
Total Frame Length:	51.00 ft
Frame Weight:	2193 lbs
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	51.66 ft
Rail Length:	180.60 in
Rail Spacing:	2.83 ft
Rail Check:	Not Checked

Support Specifications

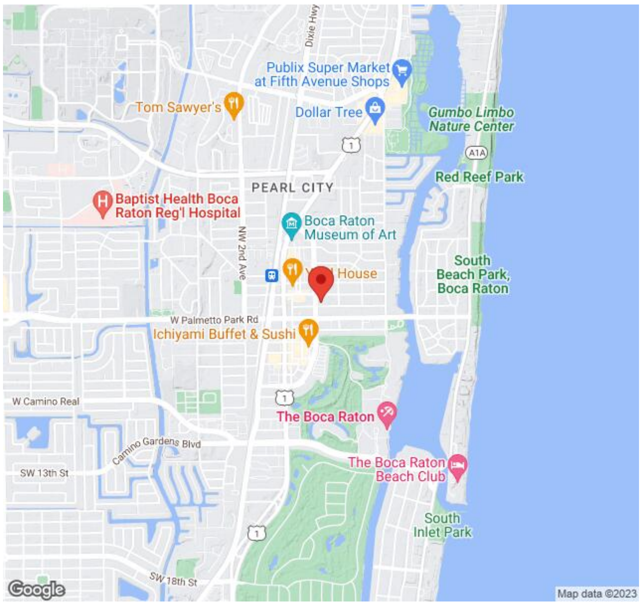
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.31 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.50 ft Pile 3: 8.00 ft
Foundation Volume:	6.414 y^3
Foundation Result:	PASSED
Mount Twist:	0.257154 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	100 NE Mizner Blvd, Boca Raton, FL 33432, USA
Wind Speed:	156 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

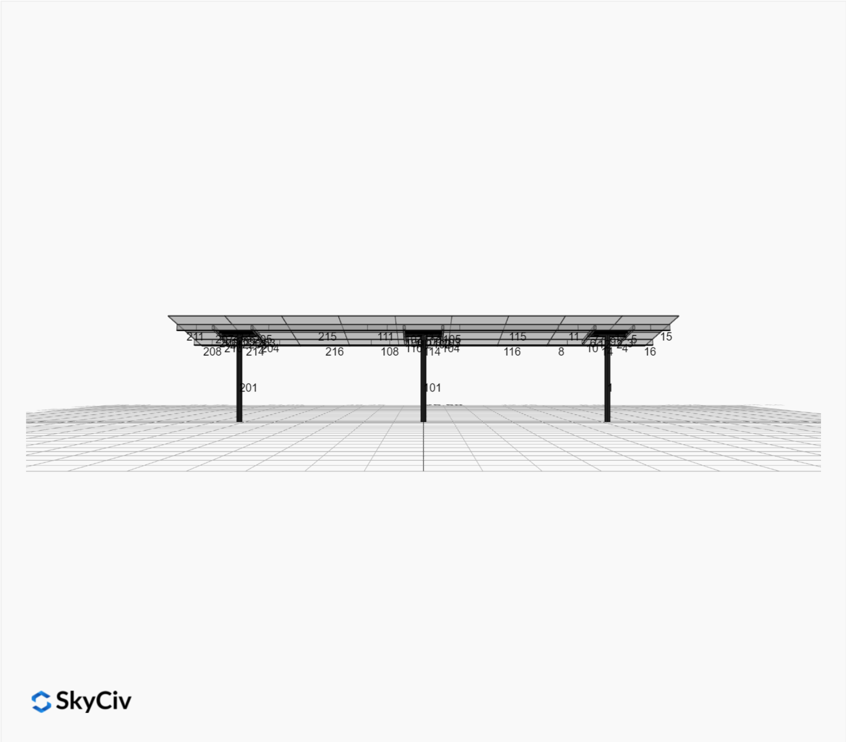
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

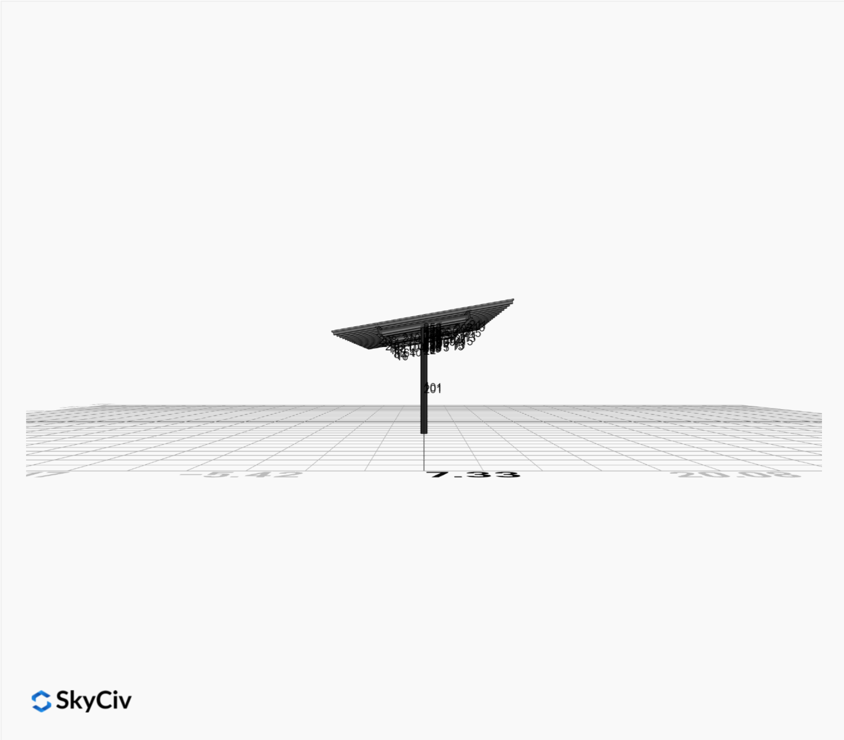
AutoDesigner Input

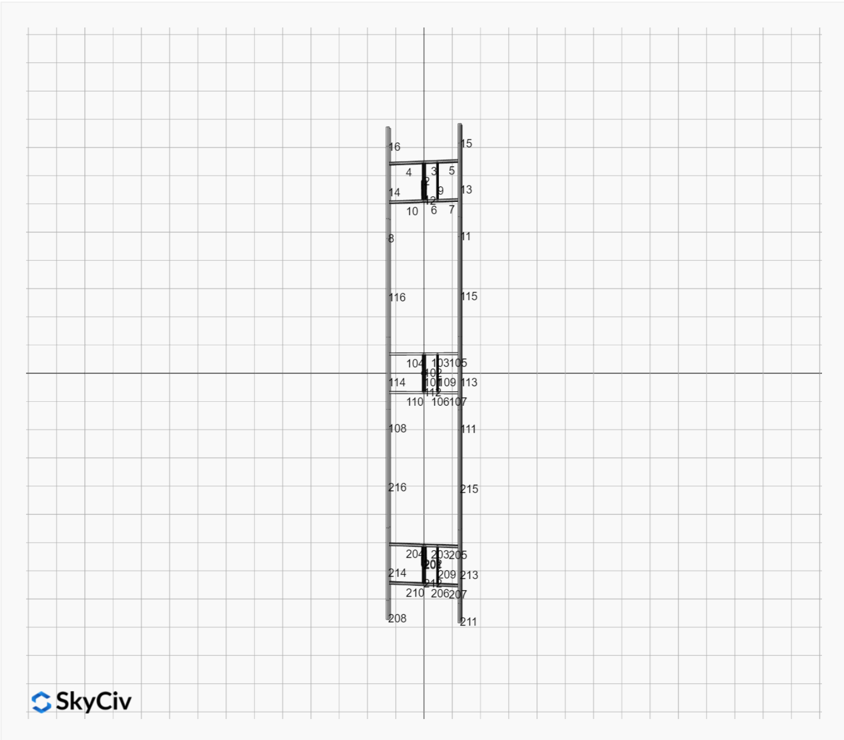

```
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"product_type":"Beam","project_id":"MTSOLAR_K8I42DL04KIK","site_address":"100 NE Mizner Blvd, Boca Raton, FL 33432, USA","module_width":44.65,"module_length":67.88,"number_rows":"4","number_columns":9,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":10,"ground_clearance":8,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"36_Round","foundation_type":"Round","foundation_size":36,"check_rails":false}
```

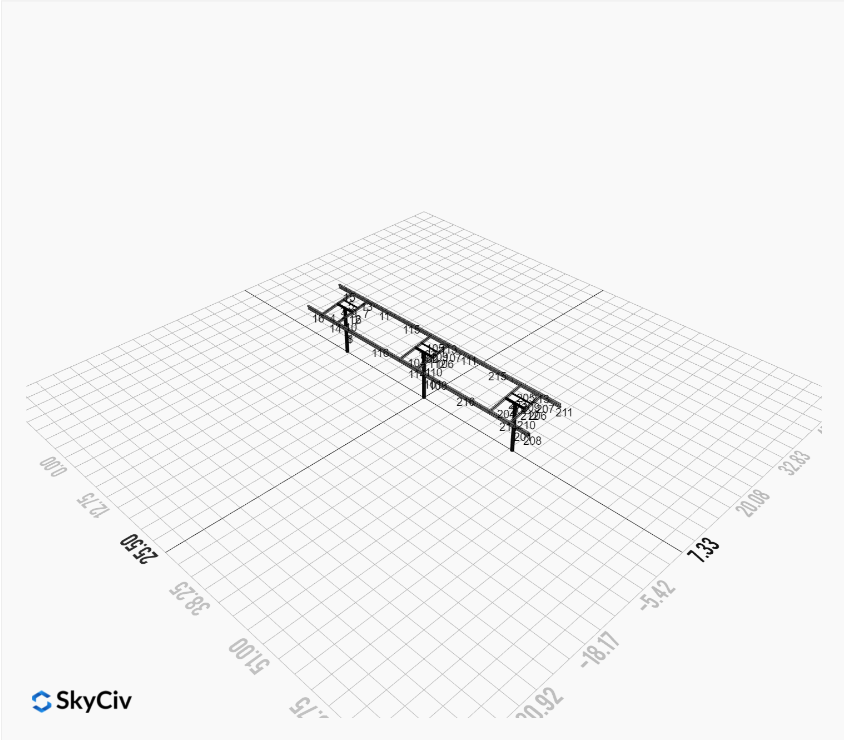

Design Notes:

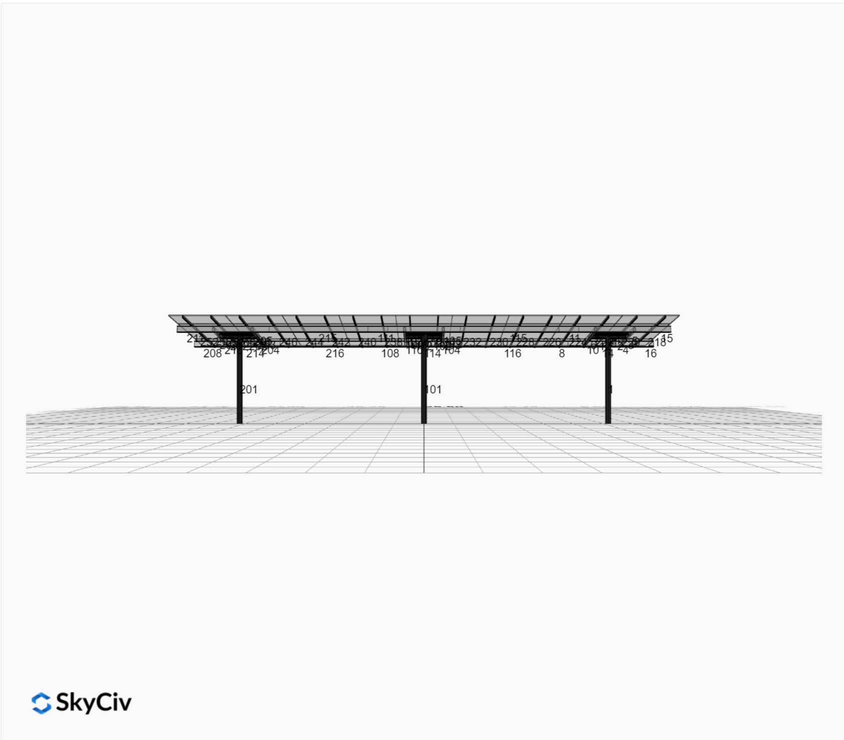
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



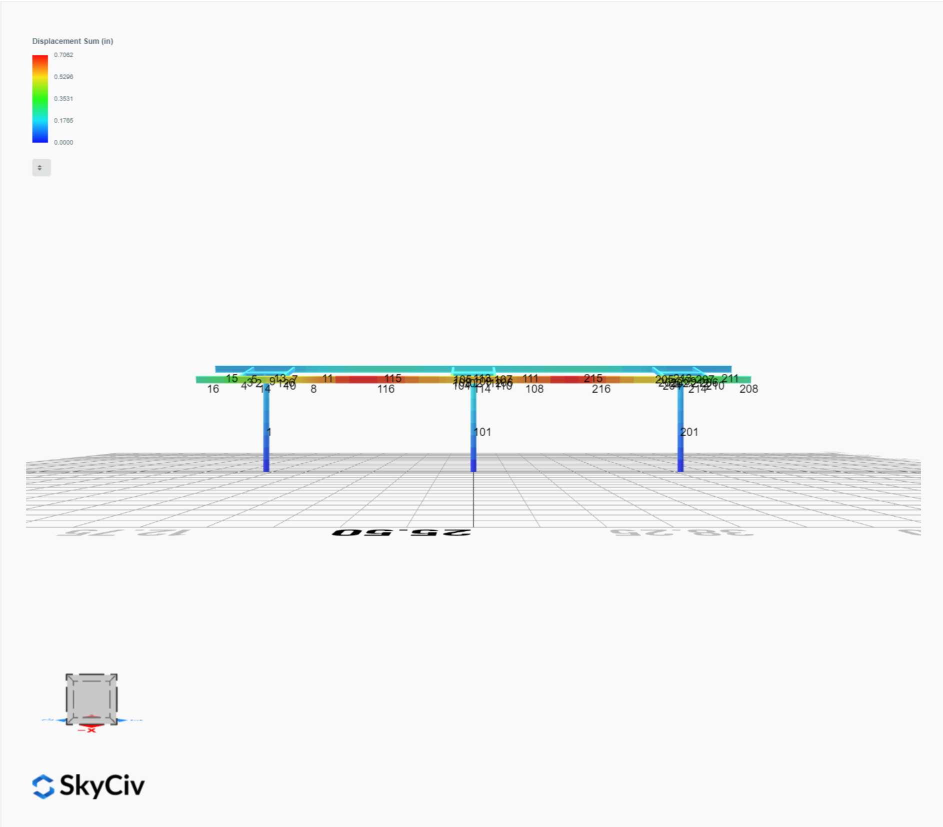

201
7.33
SkyCiv

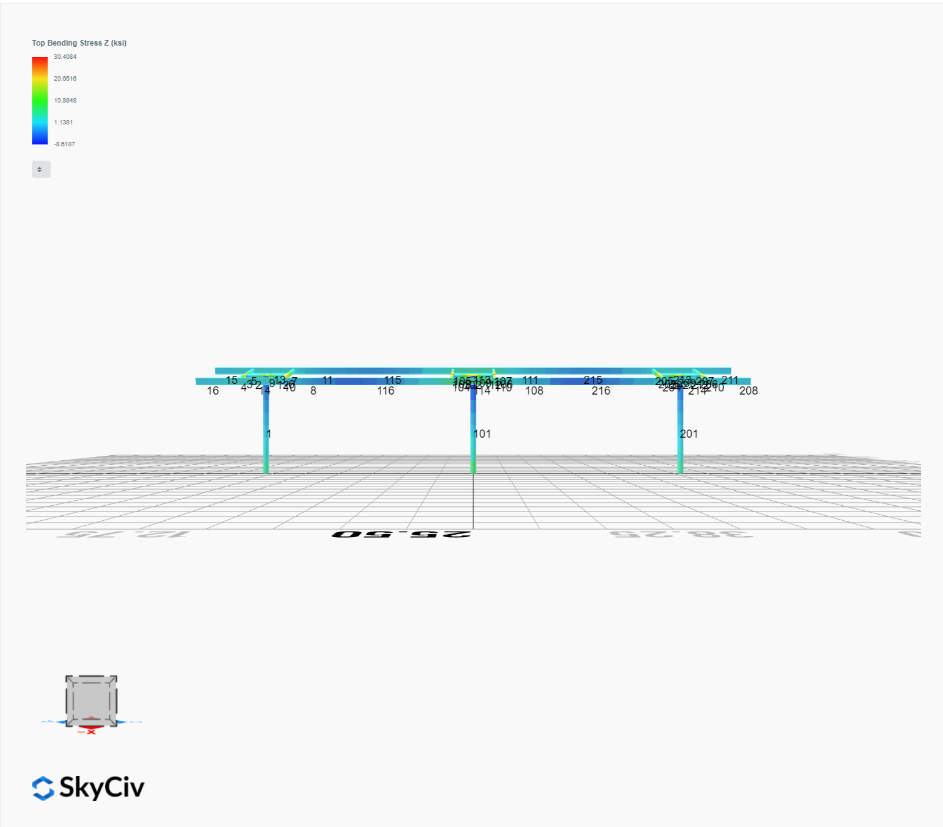

16
15
4
3
5
14
13
10
6
7
8
11
116
115
104
103105
114
109
113
110
106107
108
111
216
215
204
203205
214
209
213
210
206207
208
211
SkyCiv

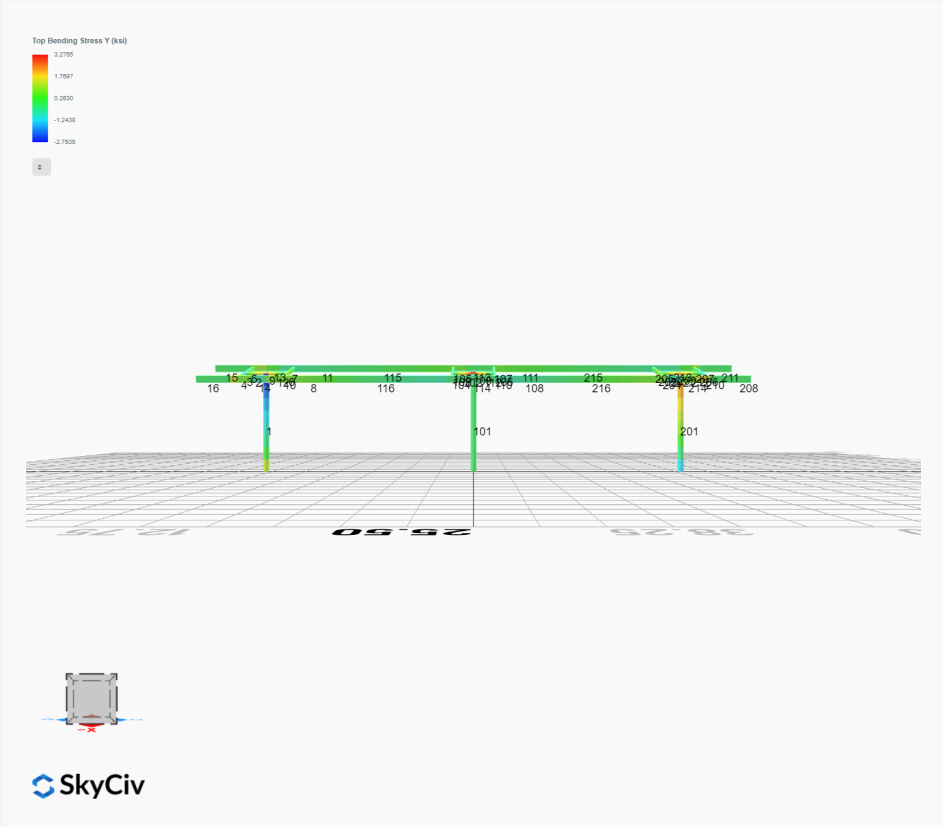

0.00
12.75
25.50
38.25
51.00
32.83
20.08
7.33
-5.42
-18.17

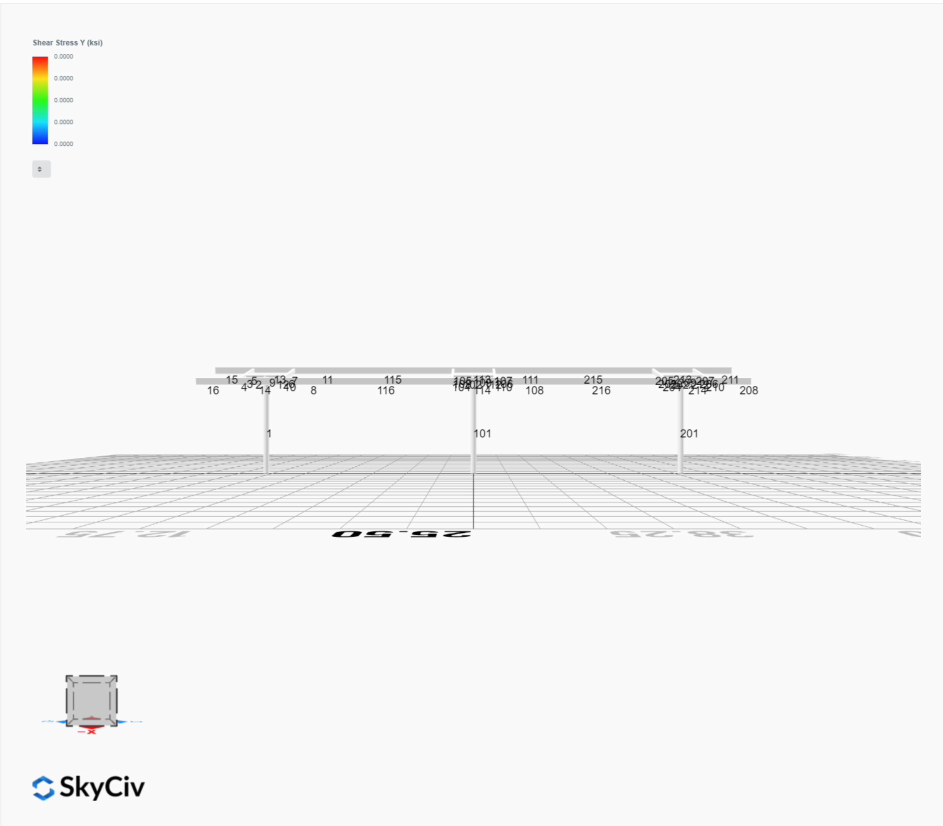


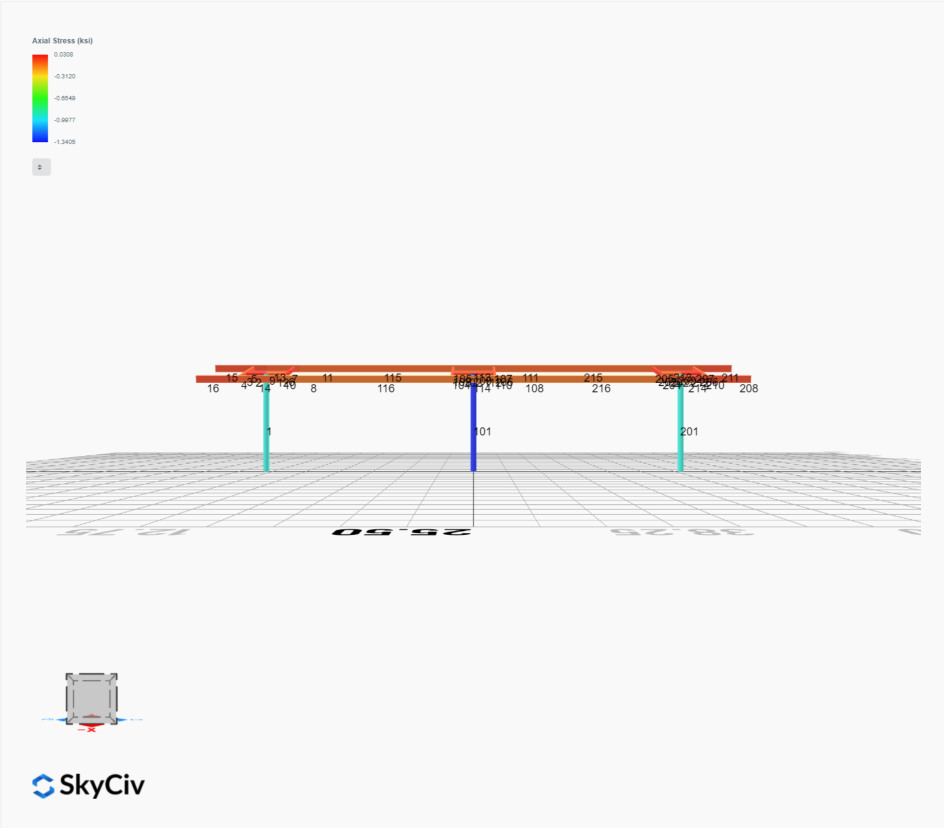
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
3.2765
1.7697
0.2630
-1.2438
-2.7505
15
16
11
8
115
116
111
108
215
216
211
208
1
101
201
25.50
SkyCiv


Shear Stress Y (ksi)
0.0000
0.0000
0.0000
0.0000
0.0000
15
16
11
8
115
116
111
108
215
216
211
208
1
101
201
25.50
SkyCiv


Axial Stress (ksi)
0.0306
-0.3120
-0.6549
-0.9977
-1.3405
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	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 2. D + L	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 3. D + (S or Lr or R)	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 3. D + (S or Lr or R)	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 5b. D + 0.7E	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0105	1.7479	0.0777	0.2188	-0.0164	-0.0522
ULS: 8. 0.6D + 0.7E	0.0063	1.0487	0.0466	0.1313	-0.0098	-0.0313
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8064	6.1824	0.3515	0.9831	-0.1581	11.7342
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8064	6.1824	0.3515	0.9831	-0.1581	11.7342
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5637	-1.3530	-0.1010	-0.2784	0.0685	-2.4814
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5339	-0.9699	-0.1027	-0.2813	0.0870	-15.4573
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6022	5.0738	0.2831	0.7920	-0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6022	5.0738	0.2831	0.7920	-0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4254	-0.5778	-0.0563	-0.1541	0.0473	-1.8741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4030	-0.2905	-0.0576	-0.1563	0.0611	-11.6060
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6022	5.0738	0.2831	0.7920	-0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6022	5.0738	0.2831	0.7920	-0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4254	-0.5778	-0.0563	-0.1541	0.0473	-1.8741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4030	-0.2905	-0.0576	-0.1563	0.0611	-11.6060
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8106	5.4832	0.3204	0.8956	-0.1516	11.7550
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8106	5.4832	0.3204	0.8956	-0.1516	11.7550
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5595	-2.0522	-0.1321	-0.3659	0.0751	-2.4605
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5296	-1.6691	-0.1338	-0.3688	0.0935	-15.4364

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	9.4887
Shear X	-1.3615
Shear Z	0.5512
Moment X	1.5428
Moment Y (Twist)	0.2571
Moment Z	26.1345

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	6.1824
Shear X	-0.8106
Shear Z	0.3515
Moment X	0.9831
Moment Y (Twist)	0.1581
Moment Z	15.4573

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	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 2. D + L	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 3. D + (S or Lr or R)	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 3. D + (S or Lr or R)	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 5b. D + 0.7E	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0210	2.3471	-0.0000	0.0000	0.0000	0.1916
ULS: 8. 0.6D + 0.7E	-0.0126	1.4083	-0.0000	0.0000	0.0000	0.1150
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0752	8.7225	0.0000	-0.0000	0.0000	14.8908
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0752	8.7225	0.0000	-0.0000	0.0000	14.8908
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7577	-2.1420	-0.0000	-0.0000	0.0000	-3.3258
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5730	-1.5223	-0.0000	0.0000	0.0000	-17.9327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8117	7.1287	0.0000	-0.0000	0.0000	11.2160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8117	7.1287	0.0000	-0.0000	0.0000	11.2160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5630	-1.0197	-0.0000	-0.0000	0.0000	-2.4465
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4245	-0.5549	-0.0000	0.0000	0.0000	-13.4017
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8117	7.1287	0.0000	-0.0000	0.0000	11.2160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8117	7.1287	0.0000	-0.0000	0.0000	11.2160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5630	-1.0197	-0.0000	-0.0000	0.0000	-2.4465
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4245	-0.5549	-0.0000	0.0000	0.0000	-13.4017
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0668	7.7837	0.0000	-0.0000	0.0000	14.8142
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0668	7.7837	0.0000	-0.0000	0.0000	14.8142
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7661	-3.0808	0.0000	-0.0000	0.0000	-3.4024
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5814	-2.4611	-0.0000	0.0000	0.0000	-18.0094

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	13.4415
Shear X	-1.7793
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	30.5051

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.7225
Shear X	-1.0752
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	18.0094

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 2. D + L	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 3. D + (S or Lr or R)	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 3. D + (S or Lr or R)	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 5b. D + 0.7E	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0105	1.7479	-0.0777	-0.2188	0.0164	-0.0522
ULS: 8. 0.6D + 0.7E	0.0063	1.0487	-0.0466	-0.1313	0.0098	-0.0313
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8064	6.1824	-0.3515	-0.9831	0.1581	11.7342
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8064	6.1824	-0.3515	-0.9831	0.1581	11.7342
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5637	-1.3530	0.1010	0.2784	-0.0685	-2.4814
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5339	-0.9699	0.1027	0.2813	-0.0869	-15.4573
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6022	5.0738	-0.2831	-0.7920	0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6022	5.0738	-0.2831	-0.7920	0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4254	-0.5778	0.0563	0.1541	-0.0473	-1.8741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4030	-0.2905	0.0576	0.1563	-0.0611	-11.6060

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6022	5.0738	-0.2831	-0.7920	0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6022	5.0738	-0.2831	-0.7920	0.1227	8.7876
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4254	-0.5778	0.0563	0.1541	-0.0473	-1.8741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4030	-0.2905	0.0576	0.1563	-0.0611	-11.6060
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8106	5.4832	-0.3204	-0.8956	0.1516	11.7550
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8106	5.4832	-0.3204	-0.8956	0.1516	11.7550
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5595	-2.0522	0.1321	0.3659	-0.0751	-2.4605
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5296	-1.6691	0.1338	0.3688	-0.0935	-15.4364

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	9.4887
Shear X	-1.3615
Shear Z	-0.5512
Moment X	-1.5428
Moment Y (Twist)	0.2572
Moment Z	26.1349

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	6.1824
Shear X	-0.8106
Shear Z	-0.3515
Moment X	-0.9831
Moment Y (Twist)	0.1581
Moment Z	15.4573

Project Details

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

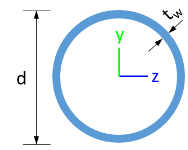
User Name: sales@mtsolar.us
Project Name: MTSOLAR_K8I42DL04KIK
Unit System: imperial

Design Input Information

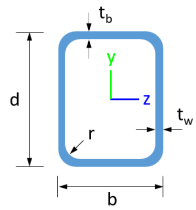
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

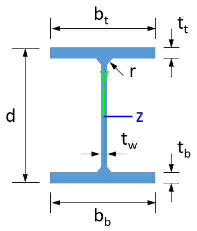
Section Dimensions



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					
7	6in Pipe Sch 40	6.63	0.28					



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

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
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	7	19.54	19.54	9.31	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.17,1.17,1.16,1.17,1.17,1.17,1.16,1.17,1.18,1.18,1.10,1.16,1.18,1.18,1.10,1.16,1.17,1.17,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.68,1.68,1.64,1.69,1.68,1.68,1.64,1.69,1.67,1.67,1.66,1.68	300	200	1
5	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.59,1.66,1.67,1.67,1.59,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.19,1.19,1.14,1.18,1.19,1.19,1.14,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.62,1.66,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.67	300	200	1
8	19	1.33	1.33	2.05	1.29,1.29,1.29,1.29,1.29,1.29,1.26,1.26,1.32,2.09,1.26,1.26,1.32,2.09,1.27,1.27,1.33,1.57,1.27,1.27,1.33,1.57,1.26,1.26,1.32,2.16	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.65,1.69,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.68	300	200	1
11	19	1.33	1.33	2.05	1.31,1.31,1.31,1.31,1.31,1.31,1.34,1.34,1.27,1.42,1.34,1.34,1.27,1.42,1.34,1.34,1.21,1.46,1.34,1.34,1.21,1.46,1.34,1.34,1.27,1.41	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.18,1.24,1.21,1.21,1.18,1.24,1.22,1.22,1.12,1.26,1.22,1.22,1.12,1.26,1.21,1.21,1.19,1.24	300	200	1
14	19	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.20,1.20,1.24,1.84,1.20,1.20,1.24,1.84,1.20,1.20,1.26,1.43,1.20,1.20,1.26,1.43,1.20,1.20,1.24,2.05	300	200	1
15	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	7	19.54	19.54	9.31	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69	300	200	1
105	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.63,1.66,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.67	300	200	1
106	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.63,1.66,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.67	30	20	1

						0	0	
108	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.31,2.31,2.09,1.11,2.31,2.31,2.09,1.11,2.34,2.34,2.08,1.29,2.34,2.34,2.08,1.29,2.31,2.31,2.09,1.06	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69	300	200	1
111	19	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,1.91,1.91,2.37,1.50,1.91,1.91,2.37,1.50,1.98,1.98,2.19,1.40,1.98,1.98,2.19,1.40,1.89,1.89,2.30,1.52	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1
113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.04,1.04,1.03,1.06,1.04,1.04,1.03,1.06,1.04,1.04,1.02,1.08,1.04,1.04,1.02,1.08,1.04,1.04,1.03,1.06	300	200	1
114	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.22,1.03,1.03,1.03,1.22,1.03,1.03,1.04,1.38,1.03,1.03,1.04,1.38,1.03,1.03,1.03,1.03	300	200	1
115	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.15,1.15,1.20,1.11,1.15,1.15,1.20,1.11,1.15,1.15,1.29,1.10,1.15,1.15,1.29,1.10,1.15,1.15,1.20,1.11	300	200	1
116	19	6.63	6.63	10.20	1.18,1.18,1.18,1.18,1.18,1.22,1.22,1.17,1.07,1.22,1.22,1.17,1.07,1.21,1.21,1.16,1.08,1.21,1.21,1.16,1.08,1.22,1.22,1.17,1.09	300	200	1
201	7	19.54	19.54	9.31	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.19,1.19,1.14,1.18,1.19,1.19,1.14,1.18,1.18,1.18,1.18,1.18	300	200	1
204	16	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.65,1.69,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.68	300	200	1
205	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.62,1.66,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.67	300	200	1
206	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.17,1.17,1.16,1.17,1.17,1.17,1.16,1.17,1.18,1.18,1.10,1.16,1.18,1.18,1.10,1.16,1.17,1.17,1.17,1.17	300	200	1
207	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.59,1.66,1.67,1.67,1.59,1.66,1.67,1.67,1.66,1.66	300	200	1
208	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.69,1.68,1.68,1.64,1.69,1.68,1.68,1.64,1.69,1.67,1.67,1.66,1.68	300	200	1
211	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.18,1.24,1.21,1.21,1.18,1.24,1.22,1.22,1.12,1.26,1.22,1.22,1.12,1.26,1.21,1.21,1.19,1.24	300	200	1
214	19	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.20,1.20,1.24,1.84,1.20,1.20,1.24,1.84,1.20,1.20,1.26,1.43,1.20,1.20,1.26,1.43,1.20,1.20,1.24,2.05	300	200	1
215	19	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.11,1.09,1.09,1.09,1.11,1.09,1.09,1.09,1.10	300	200	1
216	19	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.21,1.09,1.09,1.09,1.21,1.09,1.09,1.09,1.12,1.09,1.09,1.09,1.12,1.09,1.09,1.09,1.29	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	251.16	113.12	42.30	42.30	75.35	75.35
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	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	25.67	6.12	40.24	43.62
14	133.20	104.94	27.50	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62
101	251.16	113.12	42.30	42.30	75.35	75.35
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	126.01	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	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	104.94	23.37	6.12	40.24	43.62
114	133.20	104.94	23.60	6.12	40.24	43.62
115	133.20	69.16	17.02	6.12	40.24	43.62
116	133.20	69.16	16.56	6.12	40.24	43.62
201	251.16	113.12	42.30	42.30	75.35	75.35
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	102.39	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	102.39	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	104.94	25.67	6.12	40.24	43.62
214	133.20	104.94	27.50	6.12	40.24	43.62
215	133.20	69.16	16.87	6.12	40.24	43.62
216	133.20	69.16	16.87	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	0.084	0.618	0.085	0.018	0.007	0.636	#16	0.522	Not Required	Pass
2	0.001	0.285	0.050	0.066	0.009	0.336	#13	0.053	Not Required	Pass
3	0.001	0.552	0.013	0.054	0.002	0.566	#13	0.045	Not Required	Pass
4	0.001	0.388	0.036	0.039	0.008	0.425	#13	0.120	Not Required	Pass
5	0.001	0.341	0.013	0.055	0.004	0.354	#13	0.074	Not Required	Pass
6	0.002	0.738	0.027	0.076	0.007	0.765	#13	0.045	Not Required	Pass
7	0.002	0.458	0.028	0.074	0.006	0.469	#13	0.074	Not Required	Pass
8	0.003	0.111	0.048	0.032	0.002	0.140	#13	0.095	Not Required	Pass
9	0.002	0.082	0.028	0.003	0.002	0.111	#13	0.204	Not Required	Pass
10	0.001	0.510	0.041	0.051	0.009	0.543	#13	0.080	Not Required	Pass
11	0.002	0.144	0.046	0.050	0.002	0.167	#13	0.095	Not Required	Pass
12	0.002	0.448	0.062	0.090	0.013	0.511	#13	0.053	Not Required	Pass
13	0.002	0.125	0.075	0.065	0.003	0.162	#16	0.286	Not Required	Pass
14	0.003	0.083	0.078	0.043	0.003	0.117	#13	0.190	Not Required	Pass
15	0.000	0.022	0.004	0.018	0.001	0.025	#13	Not Required	Not Required	Pass
16	0.000	0.016	0.004	0.013	0.001	0.019	#13	Not Required	Not Required	Pass
101	0.119	0.721	0.000	0.024	0.000	0.728	#16	0.522	Not Required	Pass
102	0.001	0.533	0.080	0.111	0.015	0.613	#13	0.053	Not Required	Pass
103	0.001	0.888	0.008	0.089	0.004	0.892	#13	0.045	Not Required	Pass
104	0.001	0.679	0.031	0.068	0.006	0.697	#13	0.080	Not Required	Pass
105	0.001	0.551	0.032	0.089	0.008	0.560	#13	0.074	Not Required	Pass
106	0.001	0.888	0.008	0.089	0.004	0.892	#13	0.045	Not Required	Pass
107	0.001	0.551	0.032	0.089	0.008	0.560	#13	0.074	Not Required	Pass
108	0.003	0.061	0.044	0.046	0.002	0.107	#13	0.095	Not Required	Pass
109	0.004	0.083	0.016	0.001	0.000	0.101	#13	0.204	Not Required	Pass
110	0.001	0.679	0.031	0.068	0.006	0.697	#13	0.080	Not Required	Pass
111	0.002	0.097	0.043	0.059	0.002	0.121	#32	0.095	Not Required	Pass
112	0.001	0.533	0.080	0.111	0.015	0.613	#13	0.053	Not Required	Pass
113	0.002	0.260	0.070	0.074	0.002	0.307	#13	0.286	Not Required	Pass
114	0.005	0.267	0.073	0.058	0.002	0.315	#13	0.286	Not Required	Pass
115	0.003	0.385	0.043	0.059	0.002	0.410	#13	0.473	Not Required	Pass
116	0.005	0.260	0.044	0.046	0.002	0.279	#13	0.473	Not Required	Pass
201	0.084	0.618	0.085	0.018	0.007	0.636	#16	0.522	Not Required	Pass
202	0.002	0.448	0.062	0.090	0.013	0.511	#13	0.053	Not Required	Pass
203	0.002	0.738	0.027	0.076	0.007	0.765	#13	0.045	Not Required	Pass
204	0.001	0.510	0.041	0.051	0.009	0.543	#13	0.080	Not Required	Pass
205	0.002	0.458	0.028	0.074	0.006	0.469	#13	0.074	Not Required	Pass
206	0.001	0.552	0.013	0.054	0.002	0.566	#13	0.045	Not Required	Pass
207	0.001	0.341	0.013	0.055	0.004	0.354	#13	0.074	Not Required	Pass
208	0.000	0.016	0.004	0.013	0.001	0.019	#13	Not Required	Not Required	Pass
209	0.002	0.082	0.028	0.003	0.002	0.111	#13	0.204	Not Required	Pass
210	0.001	0.388	0.036	0.039	0.008	0.425	#13	0.120	Not Required	Pass
211	0.000	0.022	0.004	0.018	0.001	0.025	#13	Not Required	Not Required	Pass
212	0.001	0.285	0.050	0.066	0.009	0.336	#13	0.053	Not Required	Pass
213	0.002	0.125	0.075	0.065	0.003	0.162	#16	0.190	Not Required	Pass
214	0.003	0.083	0.078	0.043	0.003	0.117	#13	0.286	Not Required	Pass
215	0.003	0.411	0.046	0.050	0.002	0.433	#13	0.473	Not Required	Pass
216	0.005	0.282	0.048	0.033	0.002	0.308	#13	0.473	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

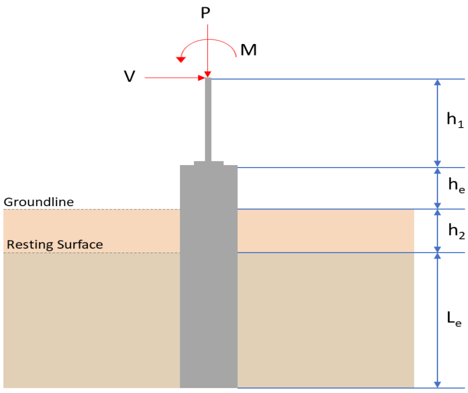
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 8\ \mathrm{ft}$ - 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)	6.182	9.489
V_x (kip)	-0.811	-1.362
V_z (kip)	0.352	0.551
M_x (kipft)	0.983	1.543
M_z (kipft)	15.457	26.135

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,

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

$$H_o = \frac{(-0.811\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.27033\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.457 \text{ kipft}) + ((-0.811 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1523 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.674 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.352 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.11733 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.983 \text{ kipft}) + ((0.352 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.32767 \text{ kipft/ft}$$

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:
$L_{e,z} = 4.4964 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.674 \text{ ft}), (4.4964 \text{ ft})]$$

$$L_{e,req} = 7.674 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.674 \text{ ft})}{(8 \text{ ft})}$$

$$Ratio = 0.95925$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.182 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.87457\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.87457\ \mathrm{kip/ft^2}\right)}{\left(2000\ \mathrm{psf}\right)}$$

$$Ratio = 0.43729$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{\left(8\ \mathrm{ft}\right)}{\left(36\ \mathrm{in}\right)}$$

$$L/D = 2.6667$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.27033\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.1523\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{\left(4 \times \left(5.1523\ \mathrm{kipft/ft}\right) \times \left(8\ \mathrm{ft}\right)\right) + \left(3 \times \left(-0.27033\ \mathrm{kip/ft}\right) \times \left(8\ \mathrm{ft}\right)^2\right)}{\left(6 \times \left(5.1523\ \mathrm{kipft/ft}\right)\right) + \left(4 \times \left(-0.27033\ \mathrm{kip/ft}\right) \times \left(8\ \mathrm{ft}\right)\right)}$$

$$a = 5.4791\ \mathrm{ft}$$

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

$$p = \frac{1.178\ \left[\left(4\, M_o\right) + \left(3\, H_o\, L_e\right)\right]^2}{L_e^2\ \left[\left(3\, M_o\right) + \left(2\, H_o\, L_e\right)\right]}$$

$$p = \frac{1.178 \times \left[\left(4 \times \left(5.1523\ \mathrm{kipft/ft}\right)\right) + \left(3 \times \left(-0.27033\ \mathrm{kip/ft}\right) \times \left(8\ \mathrm{ft}\right)\right)\right]^2}{\left(8\ \mathrm{ft}\right)^2 \times \left[\left(3 \times \left(5.1523\ \mathrm{kipft/ft}\right)\right) + \left(2 \times \left(-0.27033\ \mathrm{kip/ft}\right) \times \left(8\ \mathrm{ft}\right)\right)\right]}$$

$$p = 0.32973\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\ \left[\left(2\, M_o\right) + \left(H_o\, L_e\right)\right]}{L_e^2}$$

$$s = \frac{9.425 \times \left[\left(2 \times \left(5.1523\ \mathrm{kipft/ft}\right)\right) + \left(\left(-0.27033\ \mathrm{kip/ft}\right) \times \left(8\ \mathrm{ft}\right)\right)\right]}{\left(8\ \mathrm{ft}\right)^2}$$

$$s = 1.199\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = \left(150\ \mathrm{psf/ft}\right) \times \frac{\left(5.4791\ \mathrm{ft}\right)}{2}$$

$$p_a = 0.41093\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.32973\ \mathrm{kip/ft^2}\right)}{\left(0.41093\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.80239$$

Status: **PASS**
Ratio: **0.800**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8\ \mathrm{ft})$$

$$p_s = 1.2\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.199\ \mathrm{kip/ft^2}\right)}{\left(1.2\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9992$$

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = 0.11733\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.32767\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.32767\ \mathrm{kipft/ft}) \times (8\ \mathrm{ft})) + \left(3 \times (0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft})^2\right)}{(6 \times (0.32767\ \mathrm{kipft/ft})) + (4 \times (0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))}$$

$$a = 5.7709\ \mathrm{ft}$$

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

$$p = \frac{1.178\,[(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\,[(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.32767\ \mathrm{kipft/ft})) + (3 \times (0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]^{\,2}}{(8\ \mathrm{ft})^2 \times [(3 \times (0.32767\ \mathrm{kipft/ft})) + (2 \times (0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}$$

$$p = 0.10958\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\,[(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.32767\ \mathrm{kipft/ft})) + ((0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}{(8\ \mathrm{ft})^2}$$

$$s = 0.23474\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.7709\ \mathrm{ft})}{2}$$

$$p_a = 0.43282\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.10958\ \mathrm{kip/ft^2}\right)}{\left(0.43282\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.25319$$

Status: **PASS**
Ratio: **0.250**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8\ \mathrm{ft})$$

$$p_s = 1.2\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

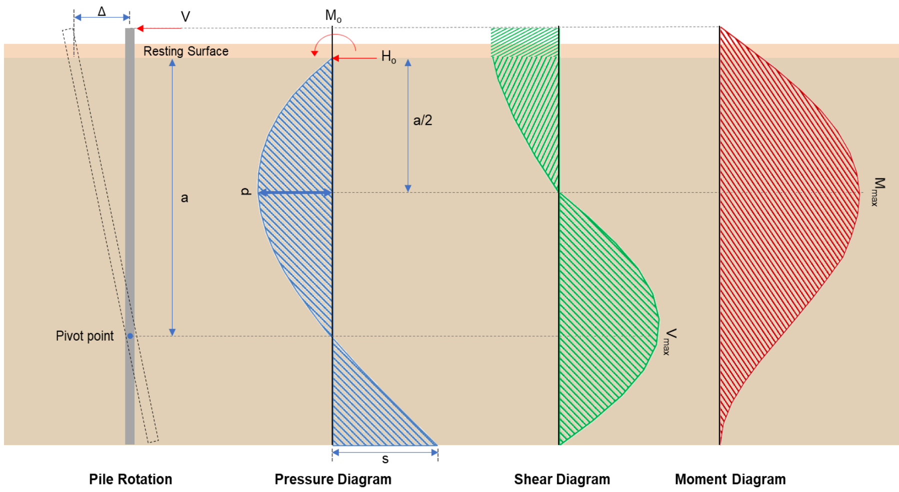
$$Ratio = \frac{s}{\ldots}$$

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

$$Ratio = \frac{(0.23474\ \text{kip/ft}^2)}{(1.2\ \text{kip/ft}^2)}$$

$$Ratio = 0.19562$$

Status: **PASS**
Ratio: **0.200**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.362\ \text{kip})}{(36\ \text{in})}$$

$$H_o = -0.454\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.135\ \text{kipft}) + ((-1.362\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 8.7117\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.7117\ \text{kipft/ft})}{(-0.454\ \text{kip/ft})}$$

$$E = 19.189\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7117\ \text{kipft/ft}) \times (8\ \text{ft})) + \left(3 \times (-0.454\ \text{kip/ft}) \times (8\ \text{ft})^2\right)}{(6 \times (8.7117\ \text{kipft/ft})) + (4 \times (-0.454\ \text{kip/ft}) \times (8\ \text{ft}))}$$

$$a = 5.4783\ \text{ft}$$

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

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

$$V_{max} = ((-0.454\ \text{kip/ft}) \times (36\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.189\ \text{ft})}{(8\ \text{ft})} + 3\right) \times \left(\frac{(5.4783\ \text{ft})}{(8\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (19.189\ \text{ft})}{(8\ \text{ft})} + 2\right) \times \left(\frac{(5.4783\ \text{ft})}{(8\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.6819 \text{ kip}$$

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

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

$$M_{max} = ((-0.454 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(19.189 \text{ ft})}{(8 \text{ ft})} + \frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right) - \left[\left(\frac{4 \times (19.189 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (19.189 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 25.734 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.551 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.18367 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.543 \text{ kipft}) + ((0.551 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.51433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.51433 \text{ kipft/ft})}{(0.18367 \text{ kip/ft})}$$

$$E = 2.8004 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.51433 \text{ kipft/ft}) \times (8 \text{ ft})) + \left(3 \times (0.18367 \text{ kip/ft}) \times (8 \text{ ft})^2\right)}{(6 \times (0.51433 \text{ kipft/ft})) + (4 \times (0.18367 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7705 \text{ ft}$$

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

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

$$V_{max} = ((0.18367 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.7705 \text{ ft})}{(8 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.7705 \text{ ft})}{(8 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.71044 \text{ kip}$$

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

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

$$M_{max} = ((0.18367 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.8004 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right) - \left[\left(\frac{4 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 2.4504 \text{ kipft}$$

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 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(9.489 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -37.077 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-37.077 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(1.8322 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.8322 \text{ in}^2\right)}{\left(1.8408 \text{ in}^2\right)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

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 center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

	Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(1017.9\ \text{in}^2\right) - \left(1.8408\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(1.8408\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 1253.9\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.489\ \text{kip})}{(1253.9\ \text{kip})}$$

$$Ratio = 0.0075675$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\ \text{in})$$

$$d = 28.8\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

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

$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500\ \text{psi})} \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,max} = 186.09\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.489\ \text{kip} \rightarrow 9489\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500\ \text{psi})} + \frac{(9489\ \text{lbf})}{6 \times \left(1017.9\ \text{in}^2\right)}\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,a} = 76.049\ \text{kip}$$

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)

$$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500\ \text{psi})} + (0.05 \times (2500\ \text{psi}))\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,b} = 204.04\ \text{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(186.09\ \text{kip}), (76.049\ \text{kip}), (204.04\ \text{kip})\right]$$

$$V_c = 76.049 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(414.72 \text{ kip}), (38.17 \text{ kip})\right]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.049 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.242 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6819 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.6819 \text{ kip})}{(74.242 \text{ kip})}$$

$$Ratio = 0.090002$$

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

Considering z-direction:

$V_{max} = 0.71044 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.71044 \text{ kip})}{(74.242 \text{ kip})}$$

$$Ratio = 0.0095691$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi\, D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

$$S_m = 4580.4\ \text{in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 4580.442in^3$$

$$\phi M_{n,1} = 62.027kipft$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(4580.4\ \text{in}^3\right)$$

$$\phi M_{n,2} = 527.23\ \text{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(62.027\ \text{kipft}), (527.23\ \text{kipft})]$$

$$\phi M_n = 62.027\ \text{kipft}$$

Considering x-direction:

$M_{max} = 25.734\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(25.734\ \text{kipft})}{(62.027\ \text{kipft})}$$

$$Ratio = 0.41489$$

Status: **PASS**
Ratio: **0.410**

Considering z-direction:

$M_{max} = 2.4504\ \text{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(2.4504\ \text{kipft})}{(62.027\ \text{kipft})}$$

$$Ratio = 0.039505$$

Status: **PASS**
Ratio: **0.040**

REFERENCES	CALCULATIONS	RESULTS

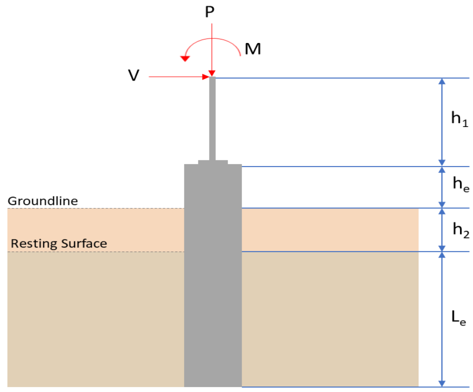
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36\ \text{in}$ - Pile diameter

$L = 8\ \text{ft}$ - 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)	6.182	9.489
V_x (kip)	-0.811	-1.362
V_z (kip)	-0.352	-0.551
M_x (kipft)	-0.983	-1.543
M_z (kipft)	15.457	26.135

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,

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

$$H_o = \frac{(-0.811\ \text{kip})}{(36\ \text{in})}$$

$$H_o = -0.27033\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.457 \text{ kipft}) + ((-0.811 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1523 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.674 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.352 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.11733 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.983 \text{ kipft}) + ((-0.352 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.32767 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.4283 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.674 \text{ ft}), (2.4283 \text{ ft})]$$

$$L_{e,req} = 7.674 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.674 \text{ ft})}{(8 \text{ ft})}$$

$$Ratio = 0.95925$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.182 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.87457\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.87457\ \mathrm{kip/ft^2})}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.43729$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8\ \mathrm{ft})}{(36\ \mathrm{in})}$$

$$L/D = 2.6667$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.27033\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.1523\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1523\ \mathrm{kipft/ft}) \times (8\ \mathrm{ft})) + \left(3 \times (-0.27033\ \mathrm{kip/ft}) \times (8\ \mathrm{ft})^2\right)}{(6 \times (5.1523\ \mathrm{kipft/ft})) + (4 \times (-0.27033\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))}$$

$$a = 5.4791\ \mathrm{ft}$$

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

$$p = \frac{1.178\,[(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\,[(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.1523\ \mathrm{kipft/ft})) + (3 \times (-0.27033\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]^2}{(8\ \mathrm{ft})^2 \times [(3 \times (5.1523\ \mathrm{kipft/ft})) + (2 \times (-0.27033\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}$$

$$p = 0.32973\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\,[(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.1523\ \mathrm{kipft/ft})) + ((-0.27033\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}{(8\ \mathrm{ft})^2}$$

$$s = 1.199\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.4791\ \mathrm{ft})}{2}$$

$$p_a = 0.41093\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.32973\ \mathrm{kip/ft^2})}{(0.41093\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.80239$$

Status: **PASS**
Ratio: **0.800**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8\ \mathrm{ft})$$

$$p_s = 1.2\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.199\ \mathrm{kip/ft^2}\right)}{\left(1.2\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9992$$

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = -0.11733\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.32767\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.32767\ \mathrm{kipft/ft}) \times (8\ \mathrm{ft})) + \left(3 \times (-0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft})^2\right)}{(6 \times (0.32767\ \mathrm{kipft/ft})) + (4 \times (-0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))}$$

$$a = 5.7709\ \mathrm{ft}$$

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

$$p = \frac{1.178\,[(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\,[(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.32767\ \mathrm{kipft/ft})) + (3 \times (-0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]^2}{(8\ \mathrm{ft})^2 \times [(3 \times (0.32767\ \mathrm{kipft/ft})) + (2 \times (-0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}$$

$$p = -0.046637\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\,[(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.32767\ \mathrm{kipft/ft})) + ((-0.11733\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))]}{(8\ \mathrm{ft})^2}$$

$$s = -0.041725\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.7709\ \mathrm{ft})}{2}$$

$$p_a = 0.43282\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.046637\ \mathrm{kip/ft^2}\right)}{\left(0.43282\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.10775$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8\ \mathrm{ft})$$

$$p_s = 1.2\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

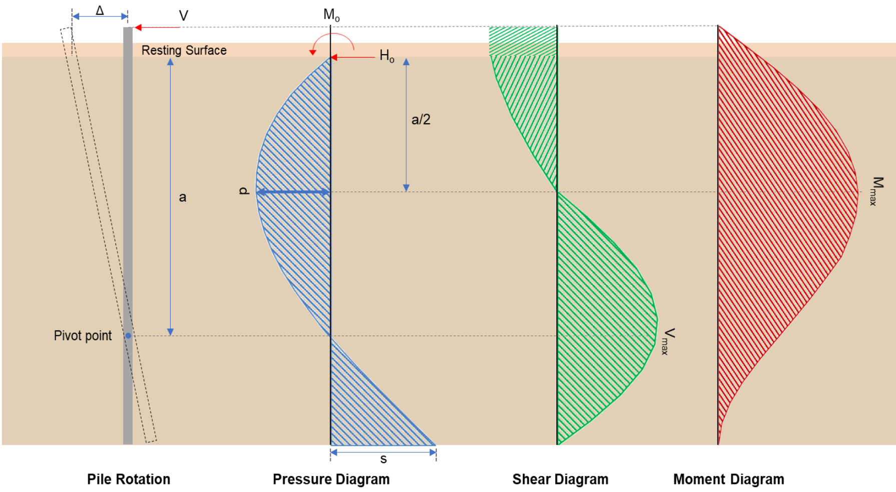
$$Ratio = \frac{s}{\ldots}$$

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

$$Ratio = \frac{\left(-0.041725\ \mathrm{kip/ft^2}\right)}{\left(1.2\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.034771$$

Status: **PASS**
Ratio: **-0.030**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.362\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.454\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.135\ \mathrm{kipft}) + ((-1.362\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 8.7117\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.7117\ \mathrm{kipft/ft})}{(-0.454\ \mathrm{kip/ft})}$$

$$E = 19.189\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7117\ \mathrm{kipft/ft}) \times (8\ \mathrm{ft})) + \left(3 \times (-0.454\ \mathrm{kip/ft}) \times (8\ \mathrm{ft})^2\right)}{(6 \times (8.7117\ \mathrm{kipft/ft})) + (4 \times (-0.454\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))}$$

$$a = 5.4783\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.454\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.189\ \mathrm{ft})}{(8\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.4783\ \mathrm{ft})}{(8\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (19.189\ \mathrm{ft})}{(8\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.4783\ \mathrm{ft})}{(8\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.6819 \text{ kip}$$

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

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

$$M_{max} = ((-0.454 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(19.189 \text{ ft})}{(8 \text{ ft})} + \frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right) - \left[\left(\frac{4 \times (19.189 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (19.189 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.4783 \text{ ft})}{2 \times (8 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 25.734 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.551 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.18367 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.543 \text{ kipft}) + ((-0.551 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.51433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.51433 \text{ kipft/ft})}{(-0.18367 \text{ kip/ft})}$$

$$E = 2.8004 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.51433 \text{ kipft/ft}) \times (8 \text{ ft})) + \left(3 \times (-0.18367 \text{ kip/ft}) \times (8 \text{ ft})^2\right)}{(6 \times (0.51433 \text{ kipft/ft})) + (4 \times (-0.18367 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7705 \text{ ft}$$

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

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

$$V_{max} = ((-0.18367 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.7705 \text{ ft})}{(8 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.7705 \text{ ft})}{(8 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.71044 \text{ kip}$$

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

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

$$M_{max} = ((-0.18367 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.8004 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right) - \left[\left(\frac{4 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 3\right) \times \left(\frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.8004 \text{ ft})}{(8 \text{ ft})} + 2\right) \times \left(\frac{(5.7705 \text{ ft})}{2 \times (8 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 2.4504 \text{ kipft}$$

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 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(9.489 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -37.077 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-37.077 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(1.8322 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.8322 \text{ in}^2\right)}{\left(1.8408 \text{ in}^2\right)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

	Main reinforcement: **6 - #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 $$\phi P_N = \phi\, 0.85\ \left[\left(0.85\, f'_{ck}\ \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(1017.9\ \text{in}^2\right) - \left(1.8408\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(1.8408\ \text{in}^2\right)\right)\right]$$ $$\phi P_N = 1253.9\ \text{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.489\ \text{kip})}{(1253.9\ \text{kip})}$$ $$Ratio = 0.0075675$$	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:** $b_w = 36\ \text{in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (36\ \text{in})$$ $$d = 28.8\ \text{in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(28.8\ \text{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.71796$$ 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 $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500\ \text{psi})} \times (36\ \text{in}) \times (28.8\ \text{in})$$ $$V_{c,max} = 186.09\ \text{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.489\ \text{kip} \rightarrow 9489\ \text{lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right] b_w\, d$$ $$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500\ \text{psi})} + \frac{(9489\ \text{lbf})}{6 \times (1017.9\ \text{in}^2)}\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$ $$V_{c,a} = 76.049\ \text{kip}$$ 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) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right] b_w\, d$$ $$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500\ \text{psi})} + (0.05 \times (2500\ \text{psi}))\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$ $$V_{c,b} = 204.04\ \text{kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(186.09\ \text{kip}), (76.049\ \text{kip}), (204.04\ \text{kip})\right]$$	

$$V_c = 76.049\ \mathrm{kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (36\ \mathrm{in}) \times (28.8\ \mathrm{in})$$

$$V_{s,a} = 414.72\ \mathrm{kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (28.8\ \mathrm{in})}{(10\ \mathrm{in})}$$

$$V_{s,b} = 38.17\ \mathrm{kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(414.72\ \mathrm{kip}), (38.17\ \mathrm{kip})\right]$$

$$V_s = 38.17\ \mathrm{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.049\ \mathrm{kip}) + (38.17\ \mathrm{kip}))$$

$$\phi V_n = 74.242\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 6.6819\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.6819\ \mathrm{kip})}{(74.242\ \mathrm{kip})}$$

$$Ratio = 0.090002$$

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

Considering z-direction:

$V_{max} = 0.71044\ \mathrm{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.71044\ \mathrm{kip})}{(74.242\ \mathrm{kip})}$$

$$Ratio = 0.0095691$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi\, D^3}{32}$$

$$S_m = \frac{\pi \times (36\ \mathrm{in})^3}{32}$$

$$S_m = 4580.4\ \mathrm{in}^3$$

$$S_m = 4580.4 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 4580.442in^3$$

$$\phi M_{n,1} = 62.027kipft$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(4580.4 \text{ in}^3\right)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 25.734 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(25.734 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.41489$$

Status: **PASS**
Ratio: **0.410**

Considering z-direction:

$M_{max} = 2.4504 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(2.4504 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.039505$$

Status: **PASS**
Ratio: **0.040**

REFERENCES	CALCULATIONS	RESULTS

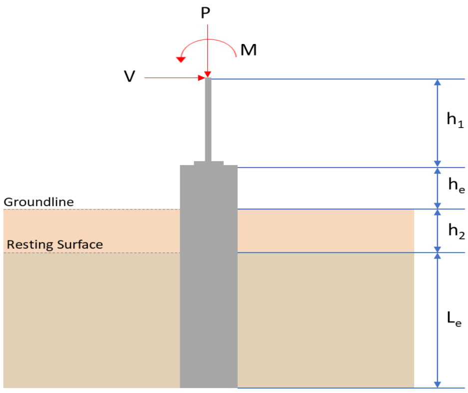
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 8.5\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.723	13.441
V_x (kip)	-1.075	-1.779
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	18.009	30.505

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,

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

$$H_o = \frac{(-1.075\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.35833\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.009 \text{ kipft}) + ((-1.075 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.003 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.875 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.875 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.875 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.875 \text{ ft})}{(8.5 \text{ ft})}$$

$$Ratio = 0.92647$$

Status: PASS
Ratio: 0.930

End-bearing Capacity (ASD)

A - Pile cros-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.723 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2341 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.2341 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.61703$$

Status: PASS
Ratio: 0.620

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.35833\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.003\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.003\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.35833\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{(6 \times (6.003\ \text{kipft/ft})) + (4 \times (-0.35833\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 5.8457\ \text{ft}$$

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

$$p = \frac{1.178\ [(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\ [(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.003\ \text{kipft/ft})) + (3 \times (-0.35833\ \text{kip/ft}) \times (8.5\ \text{ft}))]^{\ 2}}{(8.5\ \text{ft})^2 \times [(3 \times (6.003\ \text{kipft/ft})) + (2 \times (-0.35833\ \text{kip/ft}) \times (8.5\ \text{ft}))]}$$

$$p = 0.3027\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\ [(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.003\ \text{kipft/ft})) + ((-0.35833\ \text{kip/ft}) \times (8.5\ \text{ft}))]}{(8.5\ \text{ft})^2}$$

$$s = 1.1689\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(5.8457\ \text{ft})}{2}$$

$$p_a = 0.43843\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.3027\ \text{kip/ft}^2\right)}{\left(0.43843\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.69042$$

Status: **PASS**
Ratio: **0.690**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.5\ \text{ft})$$

$$p_s = 1.275\ \text{kip/ft}^2$$

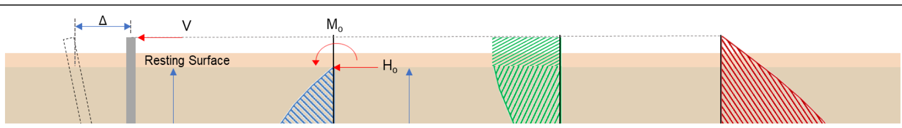
$Ratio$ - Lateral soil capacity

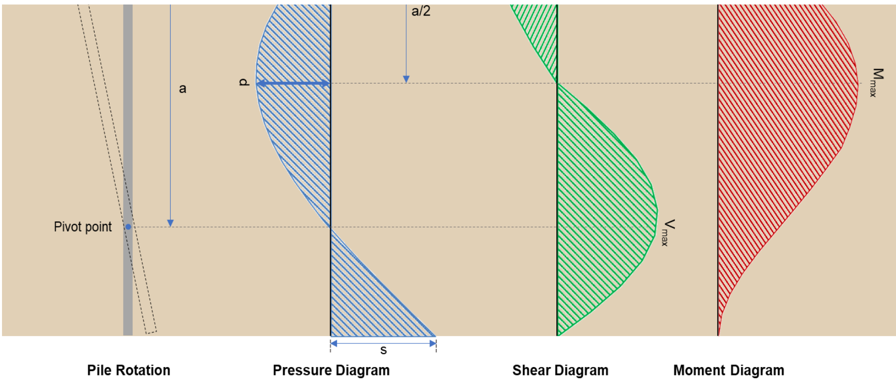
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.1689\ \text{kip/ft}^2\right)}{\left(1.275\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.91675$$

Status: **PASS**
Ratio: **0.920**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.779 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.593 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.505 \text{ kipft}) + ((-1.779 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.168 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.168 \text{ kipft/ft})}{(-0.593 \text{ kip/ft})}$$

$$E = 17.147 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.168 \text{ kipft/ft}) \times (8.5 \text{ ft})) + \left(3 \times (-0.593 \text{ kip/ft}) \times (8.5 \text{ ft})^2\right)}{(6 \times (10.168 \text{ kipft/ft})) + (4 \times (-0.593 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8426 \text{ ft}$$

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

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

$$V_{max} = ((-0.593 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.147 \text{ ft})}{(8.5 \text{ ft})} + 3\right) \times \left(\frac{(5.8426 \text{ ft})}{(8.5 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (17.147 \text{ ft})}{(8.5 \text{ ft})} + 2\right) \times \left(\frac{(5.8426 \text{ ft})}{(8.5 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 7.525 \text{ kip}$$

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

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

$$M_{max} = ((-0.593 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(17.147 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8426 \text{ ft})}{2 \times (8.5 \text{ ft})}\right) - \left[\left(\frac{4 \times (17.147 \text{ ft})}{(8.5 \text{ ft})} + 3\right) \times \left(\frac{(5.8426 \text{ ft})}{2 \times (8.5 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (17.147 \text{ ft})}{(8.5 \text{ ft})} + 2\right) \times \left(\frac{(5.8426 \text{ ft})}{2 \times (8.5 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 30.606 \text{ kipft}$$

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 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(13.441 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -36.953 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-36.953 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(1.8322 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.8322 \text{ in}^2\right)}{\left(1.8408 \text{ in}^2\right)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

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 center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

	Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$

$$\phi P_N = 1253.9\text{ kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(13.441\text{ kip})}{(1253.9\text{ kip})}$$

$$Ratio = 0.010719$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\text{ in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\text{ in})$$

$$d = 28.8\text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

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

$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,max} = 186.09\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 13.441\text{ kip} \rightarrow 13441\text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500\text{ psi})} + \frac{(13441\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,a} = 76.72\text{ kip}$$

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)

$$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,b} = 204.04\text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(186.09\text{ kip}), (76.72\text{ kip}), (204.04\text{ kip})\right]$$

$$V_c = 76.72 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(414.72 \text{ kip}), (38.17 \text{ kip})\right]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.72 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.679 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.525 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.525 \text{ kip})}{(74.679 \text{ kip})}$$

$$Ratio = 0.10077$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi\, D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 4580.442in^3$$

$$\phi M_{n,1} = 62.027kipft$$

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

	$$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(4580.4\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 527.23\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(62.027\ \mathrm{kipft}), (527.23\ \mathrm{kipft})]$$ $$\phi M_n = 62.027\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 30.606\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(30.606\ \mathrm{kipft})}{(62.027\ \mathrm{kipft})}$$ $$Ratio = 0.49343$$	Status: **PASS** Ratio: **0.490**	