Project Details

Project Name: test3

Location: 28385 Post Office Rd, Mass City, MI 49948, USA

Unique ID: 3P-17-10TOP-HD-45-L-5Hx7W-HJ61

Dealer: ______________________________

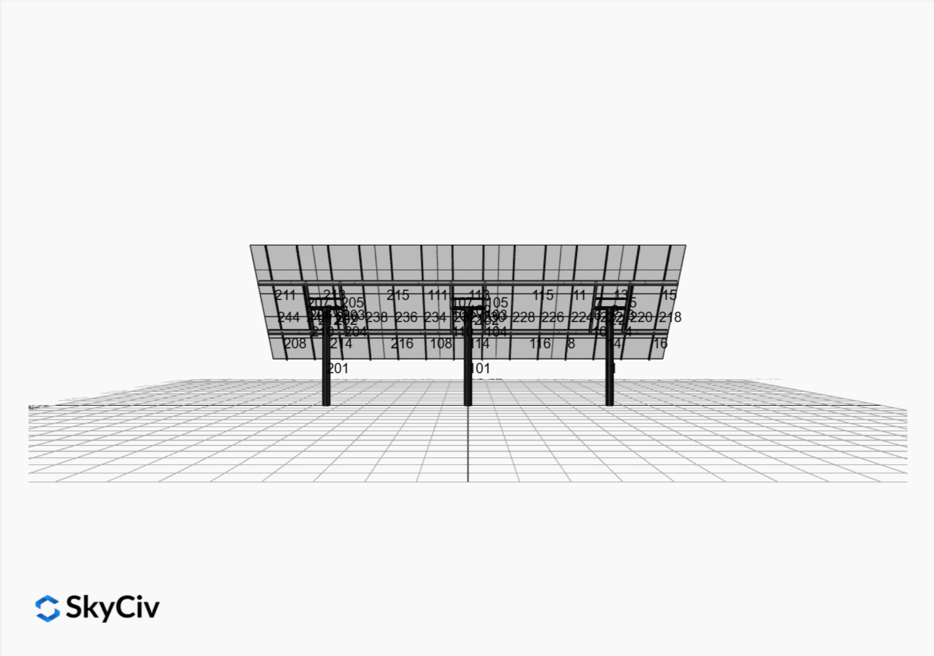
Date: Fri Oct 25 2024

Number of Modules: 35

Number of Poles: 3

Date Sold:

____



Array Dimensions N/S	17.42 ft
Array Dimensions E/W	49.53 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

MT Solar Bill of Materials (3P-17-10TOP-HD-45-L-5Hx7W-HJ61)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	7

Rail Bill of Materials

Part	Qty
Rails (207in)	14
Rail Attachment	56
Module Mid Clamp	56
Module End Clamp	28
Ground Lug	7

Site Details:

Site Address: 28385 Post Office Rd, Mass City, MI 49948, USA

Array Specification

Duty Classification:	HD
Module Width:	41.30 in
Module Length:	83.90in
Number of Rows:	5
Number of Columns:	7
Total Number of Modules:	35
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	18.34 ft
Total Frame Length:	49.00 ft
Frame Weight:	3728 lbs
Array Dimensions N/S:	17.42 ft
Array Dimensions E/W:	49.53 ft
Rail Length:	209.00 in
Rail Spacing:	3.54 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	11.67 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 6.75 ft Pile 3: 6.75 ft
Foundation Volume:	12.000 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	28385 Post Office Rd, Mass City, MI 49948, USA
Wind Speed:	125 mph
Snow Load:	80 psf

Design Disclaimer

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

AutoDesigner Input

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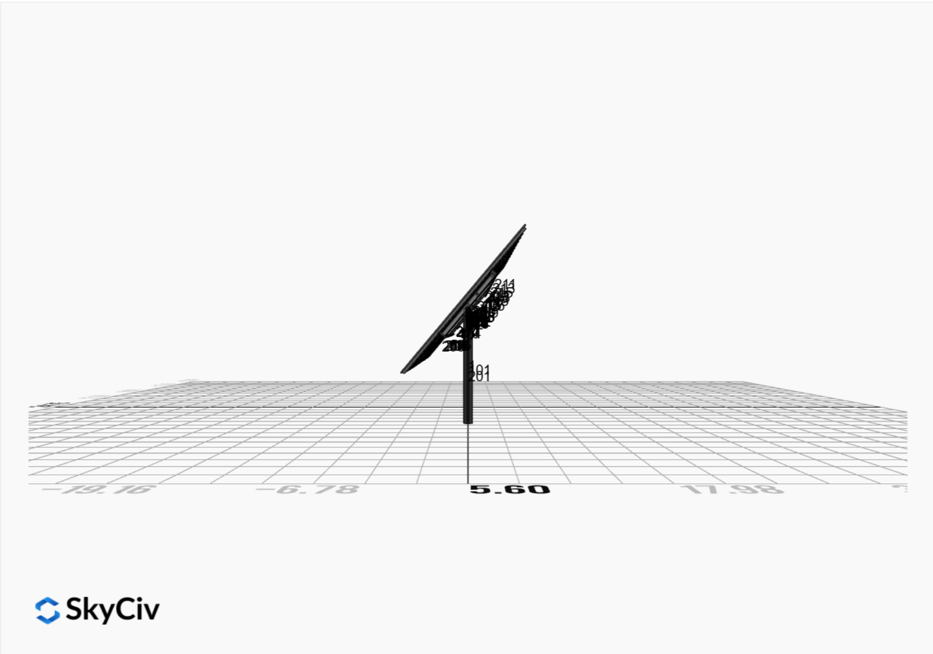
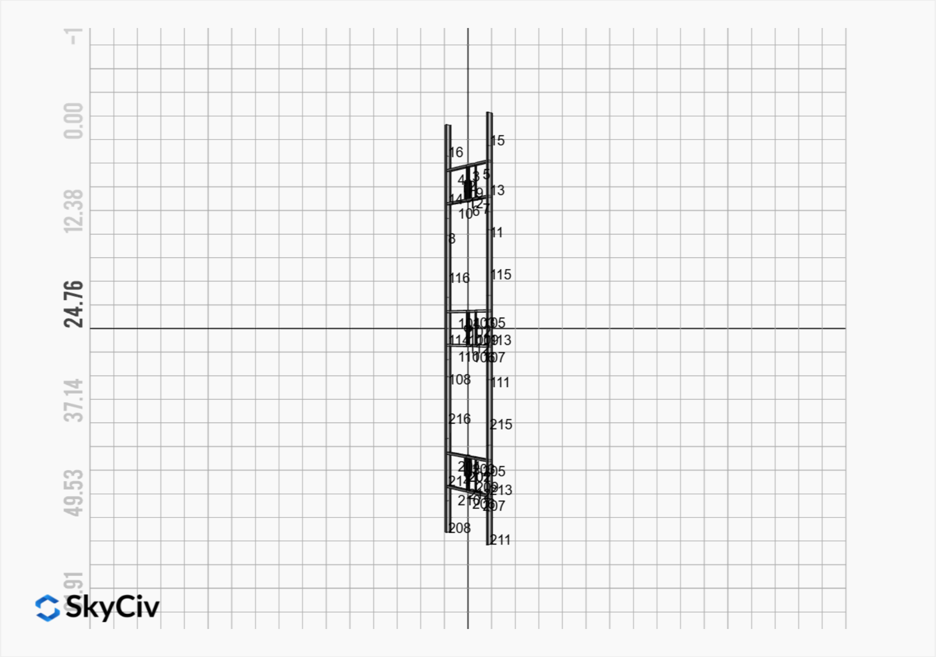
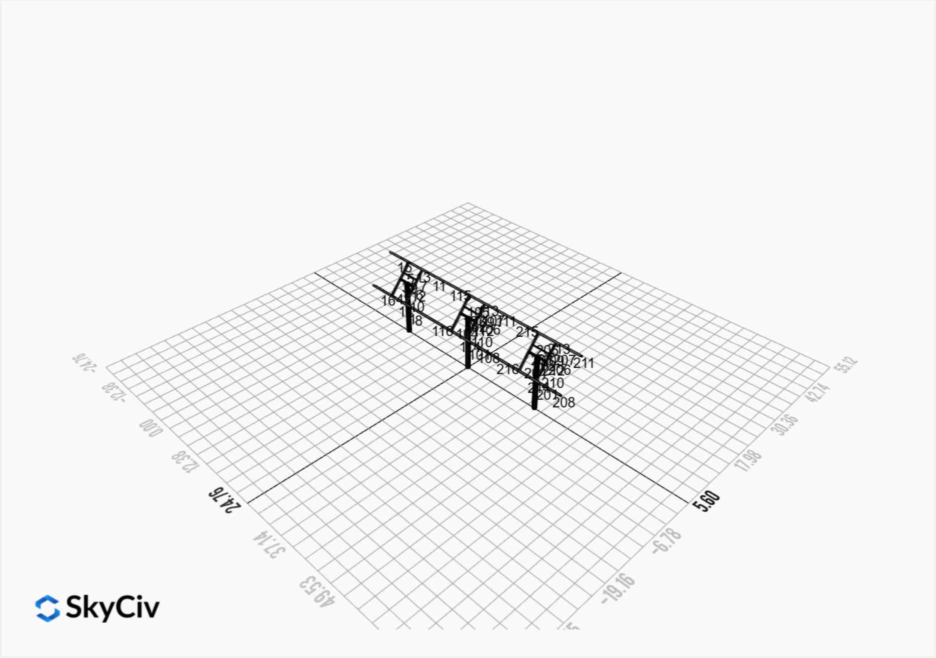
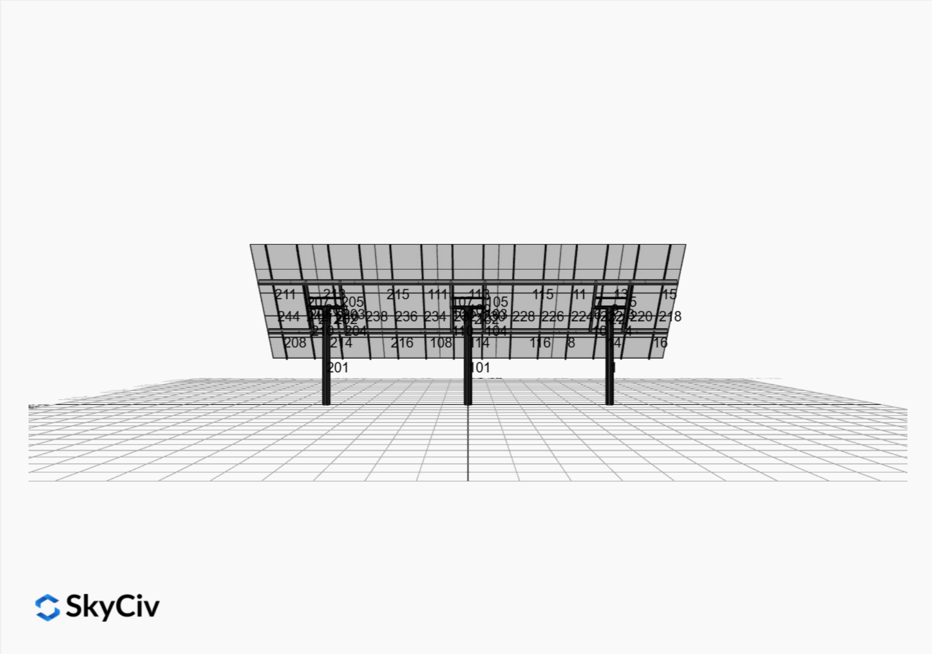
Design Notes:

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

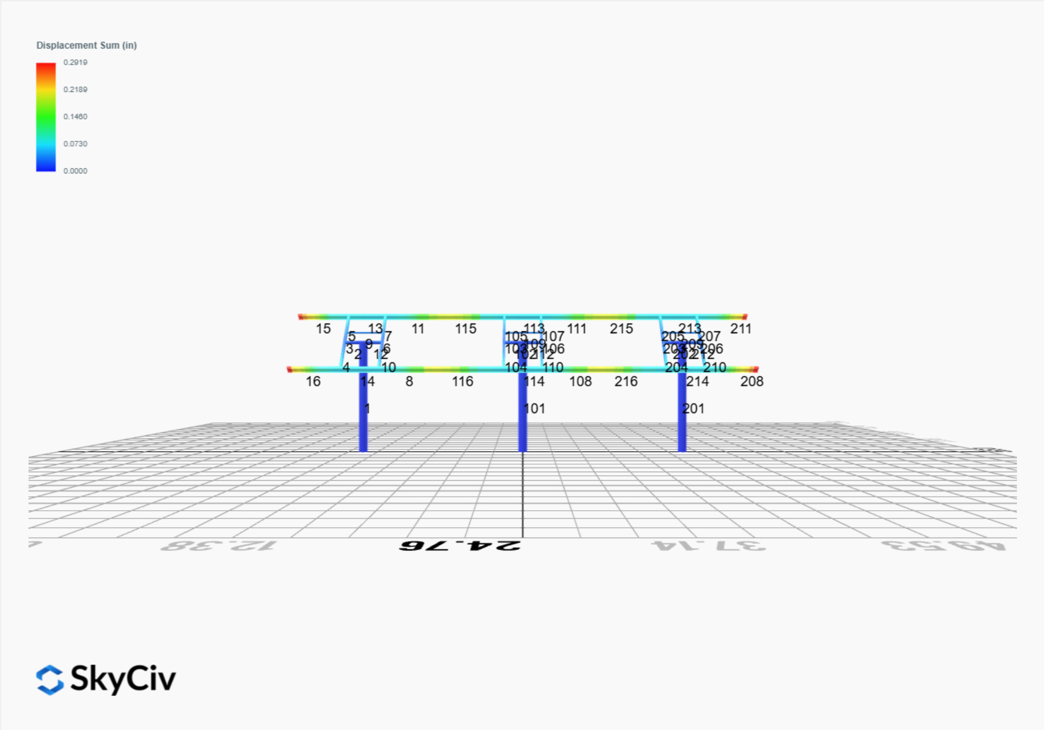


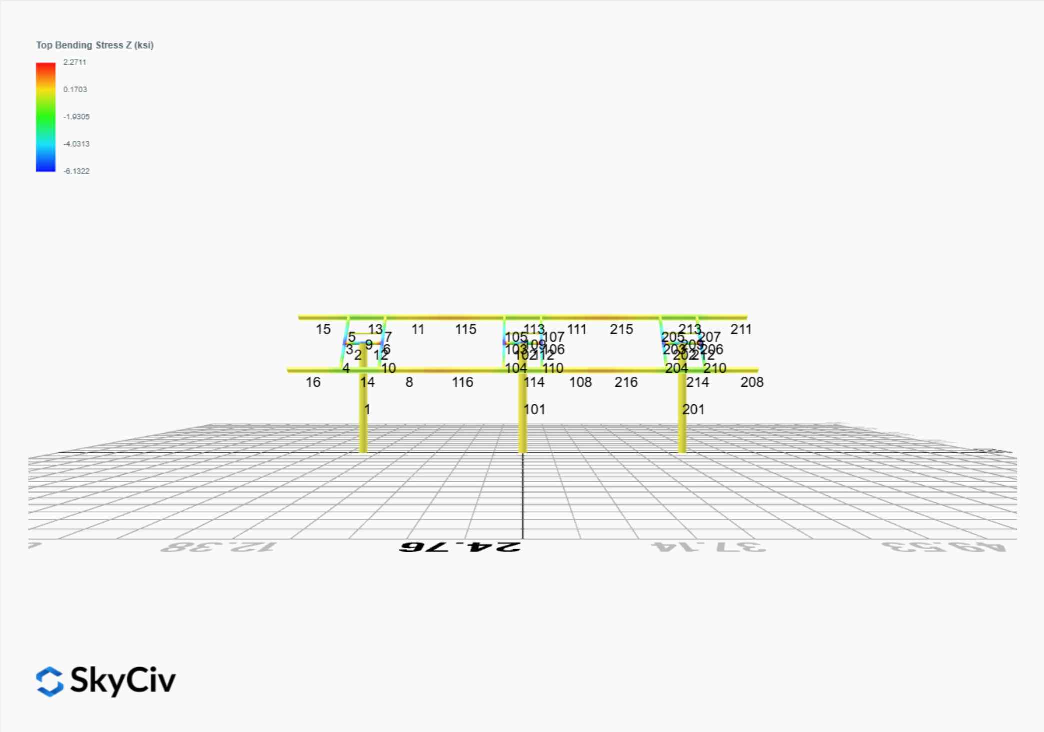


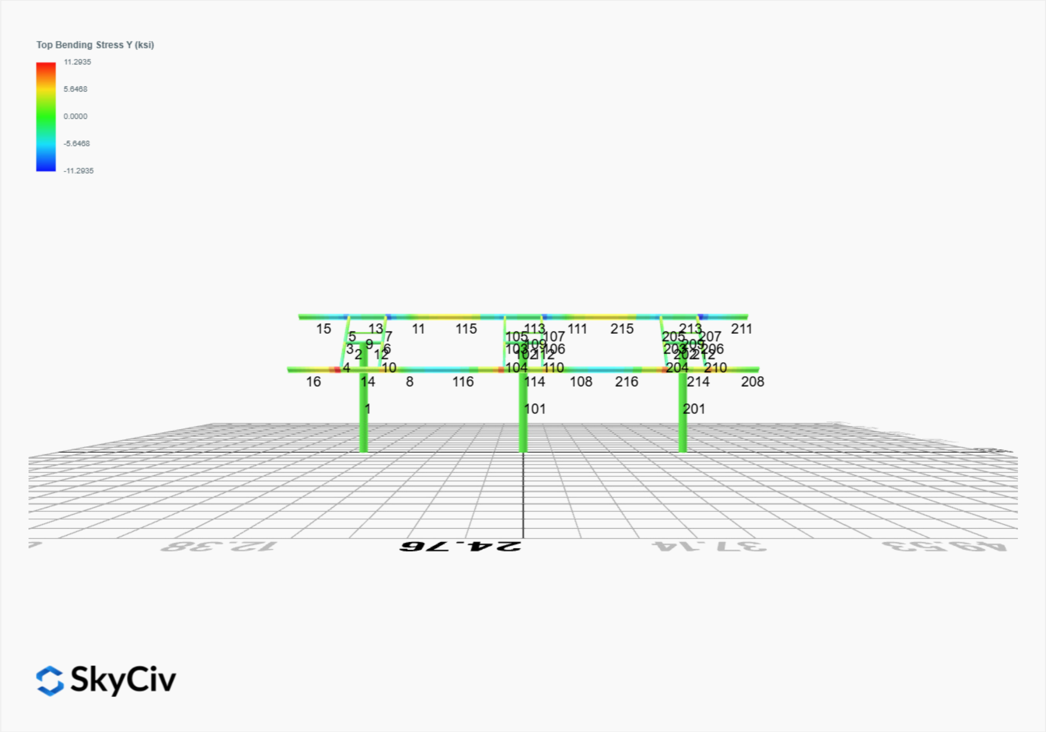


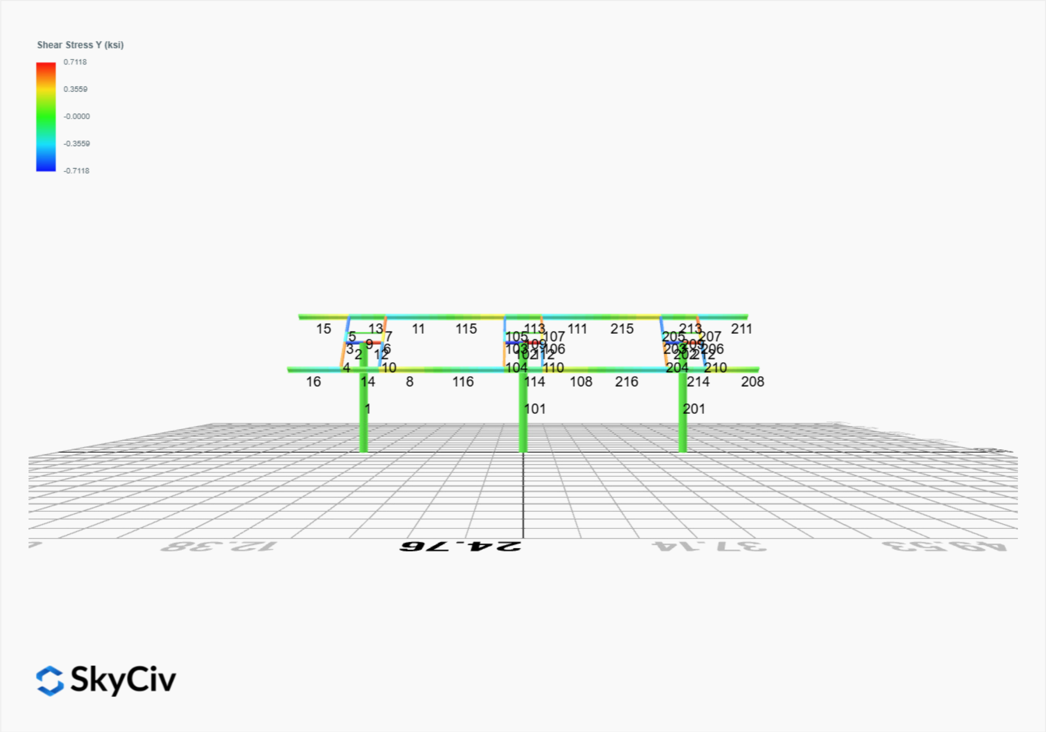


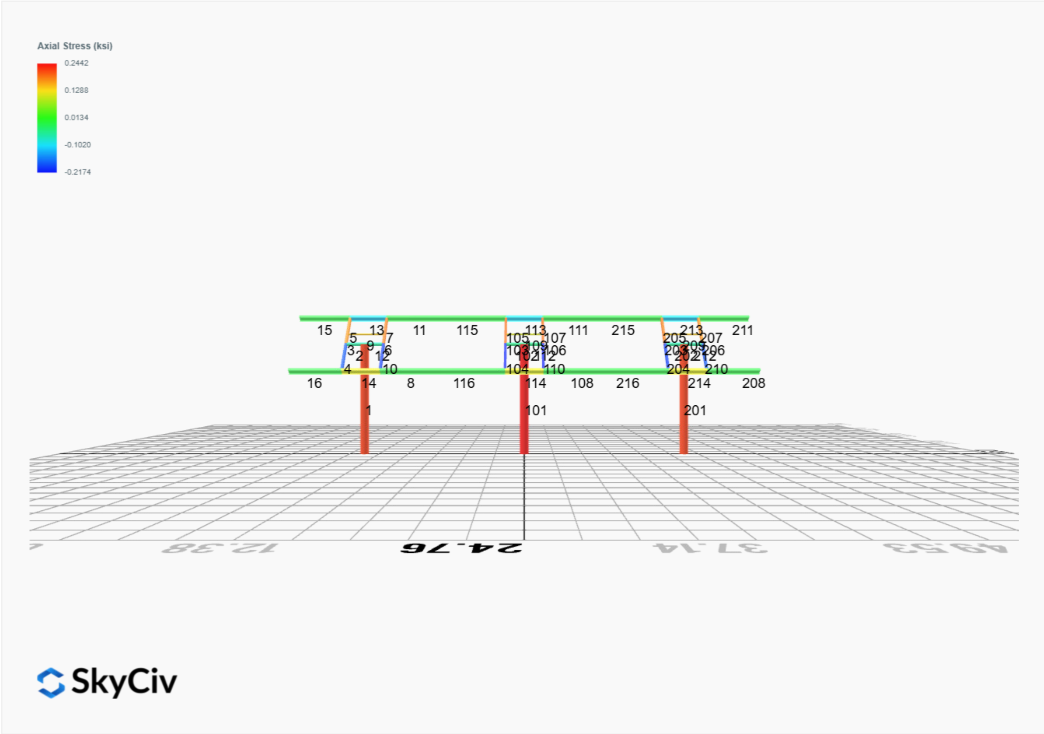
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
11.2935
5.6468
0.0000
-5.6468
-11.2935
SkyCiv


Shear Stress Y (ksi)
0.7118
0.3559
-0.0000
-0.3559
-0.7118
SkyCiv



Axial Stress (ksi)
0.2442
0.1288
0.0134
-0.1020
-0.2174
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	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 2. D + L	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 3. D + (S or Lr or R)	-0.0181	5.0453	-0.0022	0.0028	0.1481	0.2270
ULS: 3. D + (S or Lr or R)	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0152	4.3533	-0.0019	0.0022	0.1247	0.1934
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 5b. D + 0.7E	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0152	4.3533	-0.0019	0.0022	0.1247	0.1934
ULS: 8. 0.6D + 0.7E	-0.0040	1.3662	-0.0005	0.0004	0.0327	0.0557
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6620	4.5023	-0.0091	-0.0218	0.1078	31.2580
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6485	0.0518	0.0074	0.0232	0.0006	-30.7477
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0067	6.0223	-0.0081	-0.0146	0.1647	23.5673
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0152	4.3533	-0.0019	0.0022	0.1247	0.1934
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9762	2.6843	0.0043	0.0192	0.0843	-22.9370
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0152	4.3533	-0.0019	0.0022	0.1247	0.1934
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9982	3.9460	-0.0070	-0.0162	0.0945	23.4667
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9847	0.6081	0.0053	0.0176	0.0141	-23.0376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0067	2.2770	-0.0009	0.0007	0.0545	0.0929
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6593	3.5915	-0.0087	-0.0220	0.0860	31.2209
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0040	1.3662	-0.0005	0.0004	0.0327	0.0557
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6512	-0.8590	0.0077	0.0229	-0.0212	-30.7849
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0040	1.3662	-0.0005	0.0004	0.0327	0.0557

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	9.0162
Shear X	-4.4394
Shear Z	-0.0154
Moment X	0.0393
Moment Y (Twist)	0.2595
Moment Z	52.4750

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	6.0223
Shear X	-2.6620
Shear Z	-0.0091
Moment X	0.0232
Moment Y (Twist)	0.1647
Moment Z	31.2580

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	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 2. D + L	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 3. D + (S or Lr or R)	0.0362	5.2659	-0.0000	0.0000	0.0000	-0.3620
ULS: 3. D + (S or Lr or R)	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0305	4.5387	-0.0000	0.0000	0.0000	-0.3024

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 5b. D + 0.7E	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0305	4.5387	-0.0000	0.0000	0.0000	-0.3024
ULS: 8. 0.6D + 0.7E	0.0080	1.4144	-0.0000	0.0000	0.0000	-0.0743
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6953	4.6357	-0.0000	0.0000	0.0000	31.6326
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7222	0.0789	-0.0000	0.0000	0.0000	-31.5477
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0010	6.2475	-0.0000	0.0000	0.0000	23.5149
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0305	4.5387	-0.0000	0.0000	0.0000	-0.3024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0621	2.8299	-0.0000	0.0000	0.0000	-23.8703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0305	4.5387	-0.0000	0.0000	0.0000	-0.3024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0181	4.0661	-0.0000	0.0000	0.0000	23.6935
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0450	0.6485	-0.0000	0.0000	0.0000	-23.6917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0133	2.3574	-0.0000	0.0000	0.0000	-0.1238
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7006	3.6928	-0.0000	0.0000	0.0000	31.6821
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0080	1.4144	-0.0000	0.0000	0.0000	-0.0743
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7169	-0.8641	-0.0000	0.0000	0.0000	-31.4981
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0080	1.4144	-0.0000	0.0000	0.0000	-0.0743

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	9.3809
Shear X	-4.5419
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	53.0241

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	6.2475
Shear X	-2.7222
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	31.6821

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	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 2. D + L	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 3. D + (S or Lr or R)	-0.0181	5.0453	0.0022	-0.0028	-0.1481	0.2270
ULS: 3. D + (S or Lr or R)	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0152	4.3533	0.0019	-0.0022	-0.1247	0.1935
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 5b. D + 0.7E	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0152	4.3533	0.0019	-0.0022	-0.1247	0.1935
ULS: 8. 0.6D + 0.7E	-0.0040	1.3662	0.0005	-0.0004	-0.0327	0.0557
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6620	4.5023	0.0091	0.0218	-0.1078	31.2580
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6485	0.0518	-0.0074	-0.0232	-0.0006	-30.7477
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929

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	-2.0067	6.0223	0.0081	0.0146	-0.1647	23.5673
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0152	4.3533	0.0019	-0.0022	-0.1247	0.1935
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9762	2.6843	-0.0043	-0.0191	-0.0843	-22.9370
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0152	4.3533	0.0019	-0.0022	-0.1247	0.1935
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9982	3.9460	0.0070	0.0162	-0.0945	23.4667
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9847	0.6081	-0.0053	-0.0176	-0.0141	-23.0376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0067	2.2770	0.0009	-0.0007	-0.0545	0.0929
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6593	3.5915	0.0087	0.0220	-0.0860	31.2209
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0040	1.3662	0.0005	-0.0004	-0.0327	0.0557
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6512	-0.8590	-0.0077	-0.0229	0.0212	-30.7849
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0040	1.3662	0.0005	-0.0004	-0.0327	0.0557

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	9.0163
Shear X	-4.4395
Shear Z	0.0154
Moment X	-0.0393
Moment Y (Twist)	0.2593
Moment Z	52.4756

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	6.0223
Shear X	-2.6620
Shear Z	0.0091
Moment X	-0.0232
Moment Y (Twist)	0.1647
Moment Z	31.2580

Project Details

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

Design Input Information

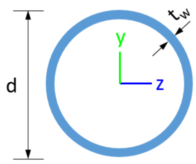
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

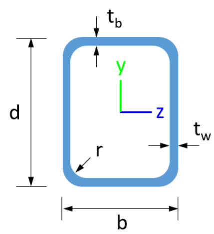
Design Materials

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

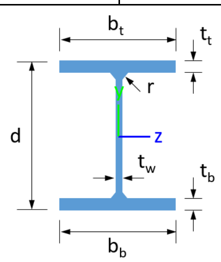
Section Dimensions



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					
11	10in Pipe Sch 40	10.75	0.36					



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
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties

Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	11	24.51	24.51	11.67	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
8	19	1.33	1.33	2.05	1.90,1.90,1.90,1.91,1.90,1.90,1.90,1.90,1.90,1.90,1.90,1.90,1.90,1.90,1.90,1.91,1.91,1.91,1.90,1.90,1.91,1.90,1.90,1.90,1.90,1.90	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
11	19	1.33	1.33	2.05	1.89,1.90,1.89,1.90,1.90,1.89,1.92,1.90,1.94,1.90,1.92,1.89,1.93,1.89,1.91,1.90,1.84,1.90,1.91,1.89,1.94,1.89,1.92,1.89,1.93,1.89	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12	300	200	1
14	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.12	300	200	1
15	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	11	24.51	24.51	11.67	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
104	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
108	19	1.33	1.33	2.05	1.59,1.60,1.59,1.60,1.59,1.59,1.56,1.60,1.53,1.60,1.56,1.59,1.54,1.59,1.58,1.60,1.73,1.60,1.56,1.59,1.53,1.59,1.56,1.59,1.55,1.59	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
111	19	1.33	1.33	2.05	1.65,1.66,1.65,1.66,1.66,1.65,1.56,1.66,1.47,1.66,1.55,1.65,1.50,1.65,1.60,1.66,2.04,1.66,1.56,1.65,1.46,1.65,1.54,1.65,1.51,1.65	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1

113	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.07,1.06,1.06,1.06,1.07,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.07,1.06,1.06,1.06,1.07,1.06	300	200	1
114	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06	300	200	1
115	19	4.84	4.84	7.45	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.13,1.11,1.13,1.12,1.13,1.11,1.13,1.13,1.13,1.18,1.13,1.12,1.13,1.11,1.13,1.12,1.13,1.11,1.13	300	200	1
116	19	4.84	4.84	7.45	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.14,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.12,1.13	300	200	1
201	11	24.51	24.51	11.67	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
204	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
208	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
211	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12	300	200	1
214	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12	300	200	1
215	19	4.84	4.84	7.45	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	300	200	1
216	19	4.84	4.84	7.45	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	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	535.87	335.40	147.68	147.68	160.76	160.76
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	123.95	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	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	25.57	6.12	40.24	43.62
14	133.20	85.85	25.55	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	535.87	335.40	147.68	147.68	160.76	160.76

101	535.87	335.40	147.68	147.68	160.76	160.76
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	123.95	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	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.97	6.12	40.24	43.62
114	133.20	85.85	24.12	6.12	40.24	43.62
115	133.20	86.20	25.46	6.12	40.24	43.62
116	133.20	86.20	25.72	6.12	40.24	43.62
201	535.87	335.40	147.68	147.68	160.76	160.76
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	52.83	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	52.83	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	25.57	6.12	40.24	43.62
214	133.20	85.85	25.55	6.12	40.24	43.62
215	133.20	86.20	26.41	6.12	40.24	43.62
216	133.20	86.20	26.64	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	0.027	0.355	0.001	0.028	0.000	0.367	#13	0.400	Not Required	Pass
2	0.002	0.294	0.206	0.071	0.038	0.476	#13	0.035	Not Required	Pass
3	0.011	0.480	0.071	0.048	0.015	0.522	#13	0.045	Not Required	Pass
4	0.010	0.478	0.148	0.048	0.032	0.533	#13	0.080	Not Required	Pass
5	0.010	0.298	0.151	0.048	0.037	0.315	#13	0.074	Not Required	Pass
6	0.011	0.471	0.074	0.047	0.015	0.513	#13	0.045	Not Required	Pass
7	0.011	0.292	0.146	0.047	0.037	0.312	#13	0.074	Not Required	Pass
8	0.000	0.045	0.100	0.030	0.016	0.140	#21	0.095	Not Required	Pass
9	0.009	0.032	0.042	0.001	0.000	0.077	#13	0.204	Not Required	Pass
10	0.011	0.468	0.142	0.047	0.032	0.527	#13	0.080	Not Required	Pass
11	0.000	0.044	0.099	0.030	0.017	0.138	#21	0.095	Not Required	Pass
12	0.002	0.289	0.197	0.071	0.037	0.460	#13	0.035	Not Required	Pass
13	0.006	0.155	0.374	0.041	0.023	0.501	#21	0.286	Not Required	Pass
14	0.009	0.158	0.374	0.041	0.023	0.501	#21	0.190	Not Required	Pass
15	0.000	0.052	0.174	0.023	0.013	0.220	#21	Not Required	Not Required	Pass

16	0.000	0.052	0.174	0.023	0.013	0.220	#21	Not Required	Not Required	Pass
101	0.028	0.359	0.000	0.028	0.000	0.371	#13	0.400	Not Required	Pass
102	0.001	0.302	0.201	0.074	0.038	0.475	#13	0.035	Not Required	Pass
103	0.011	0.487	0.082	0.049	0.020	0.534	#13	0.045	Not Required	Pass
104	0.011	0.485	0.136	0.049	0.030	0.542	#13	0.080	Not Required	Pass
105	0.011	0.302	0.138	0.048	0.034	0.319	#13	0.074	Not Required	Pass
106	0.011	0.487	0.082	0.049	0.020	0.534	#13	0.045	Not Required	Pass
107	0.011	0.302	0.138	0.048	0.034	0.319	#13	0.074	Not Required	Pass
108	0.000	0.052	0.101	0.028	0.016	0.146	#21	0.095	Not Required	Pass
109	0.006	0.028	0.040	0.001	0.000	0.069	#13	0.204	Not Required	Pass
110	0.011	0.485	0.136	0.049	0.030	0.542	#13	0.080	Not Required	Pass
111	0.000	0.053	0.100	0.028	0.016	0.144	#21	0.095	Not Required	Pass
112	0.001	0.302	0.201	0.074	0.038	0.475	#13	0.035	Not Required	Pass
113	0.006	0.113	0.333	0.038	0.023	0.421	#21	0.286	Not Required	Pass
114	0.008	0.116	0.332	0.038	0.023	0.419	#21	0.286	Not Required	Pass
115	0.000	0.095	0.189	0.028	0.016	0.273	#21	0.346	Not Required	Pass
116	0.000	0.096	0.190	0.028	0.016	0.275	#21	0.346	Not Required	Pass
201	0.027	0.355	0.001	0.028	0.000	0.367	#13	0.400	Not Required	Pass
202	0.002	0.289	0.197	0.071	0.037	0.460	#13	0.035	Not Required	Pass
203	0.011	0.471	0.074	0.047	0.015	0.513	#13	0.045	Not Required	Pass
204	0.011	0.468	0.142	0.047	0.032	0.527	#13	0.080	Not Required	Pass
205	0.011	0.292	0.146	0.047	0.037	0.312	#13	0.074	Not Required	Pass
206	0.011	0.480	0.071	0.048	0.015	0.522	#13	0.045	Not Required	Pass
207	0.010	0.298	0.151	0.048	0.037	0.315	#13	0.074	Not Required	Pass
208	0.000	0.052	0.174	0.023	0.013	0.220	#21	Not Required	Not Required	Pass
209	0.009	0.032	0.042	0.001	0.000	0.077	#13	0.204	Not Required	Pass
210	0.010	0.478	0.148	0.048	0.032	0.533	#13	0.080	Not Required	Pass
211	0.000	0.052	0.174	0.023	0.013	0.220	#21	Not Required	Not Required	Pass
212	0.002	0.294	0.206	0.071	0.038	0.476	#13	0.035	Not Required	Pass
213	0.006	0.155	0.374	0.041	0.023	0.501	#21	0.190	Not Required	Pass
214	0.009	0.158	0.374	0.041	0.023	0.501	#21	0.286	Not Required	Pass
215	0.000	0.093	0.189	0.030	0.017	0.270	#21	0.346	Not Required	Pass
216	0.000	0.093	0.190	0.030	0.016	0.272	#21	0.346	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

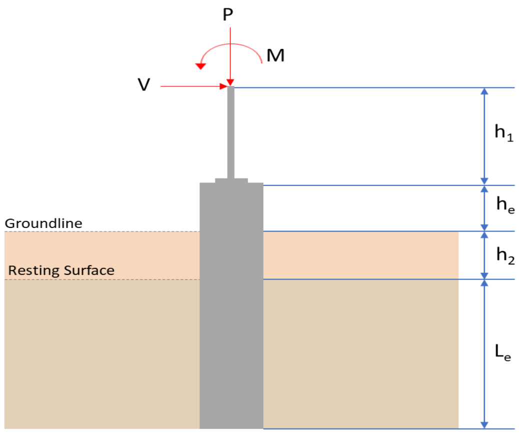
SkyCiv Foundation Design

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
$L = 6.75\ \mathrm{ft}$ - 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)	6.022	9.016
V_x (kip)	-2.662	-4.439
V_z (kip)	-0.009	-0.015
M_x (kipft)	0.023	0.039
M_z (kipft)	31.258	52.475

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,

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

$$H_o = \frac{(-2.662\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42389\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.258\ \text{kipft}) + ((-2.662\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.9774\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.2154\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.009\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.0014331\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.023\ \text{kipft}) + ((-0.009\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.0036624\ \text{kipft/ft}$$

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:
$L_{e,z} = 0.62103\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2154\ \text{ft}), (0.62103\ \text{ft})]$$

$$L_{e,req} = 6.215\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.215\ \text{ft})}{(6.75\ \text{ft})}$$

$$Ratio = 0.92074$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.022\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.37637\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.37637\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.18819$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.42389\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.9774\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9774\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.42389\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (4.9774\ \text{kipft/ft})) + (4 \times (-0.42389\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6558\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9774\ \text{kipft/ft})) + (3 \times (-0.42389\ \text{kip/ft}) \times (6.75\ \text{ft}))]^{\ 2}}{(6.75\ \text{ft})^2 \times [(3 \times (4.9774\ \text{kipft/ft})) + (2 \times (-0.42389\ \text{kip/ft}) \times (6.75\ \text{ft}))]}$$

$$p = 0.22927\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.9774\ \text{kipft/ft})) + ((-0.42389\ \text{kip/ft}) \times (6.75\ \text{ft}))]}{(6.75\ \text{ft})^2}$$

$$s = 0.93413\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.6558\ \text{ft})}{2}$$

$$p_a = 0.34919\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

	$$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(0.22927 \text{ kip/ft}^2\right)}{\left(0.34919 \text{ kip/ft}^2\right)}$$ $$Ratio = 0.65659$$	Status: **PASS** Ratio: **0.660**	
	p_s - Allowable lateral soil pressure at depth L_e, $$p_s = R\, L_e$$ $$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$ $$p_s = 1.0125 \text{ kip/ft}^2$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$ $$Ratio = \frac{\left(0.93413 \text{ kip/ft}^2\right)}{\left(1.0125 \text{ kip/ft}^2\right)}$$ $$Ratio = 0.9226$$	Status: **PASS** Ratio: **0.920**	
	Considering z-direction: $H_o = -0.0014331 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0036624 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $$a = \frac{(4\, M_o\, L_e) + \left(3\, H_o\, L_e^2\right)}{(6\, M_o) + (4\, H_o\, L_e)}$$ $$a = \frac{(4 \times (0.0036624 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.0036624 \text{ kipft/ft})) + (4 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$ $$a = 4.8588 \text{ ft}$$ p - Earth pressure against the pile at distance a/2 from resting surface, $$p = \frac{0.75\, [(4\, M_o) + (3\, H_o\, L_e)]^2}{L_e^2\, [(3\, M_o) + (2\, H_o\, L_e)]}$$ $$p = \frac{0.75 \times [(4 \times (0.0036624 \text{ kipft/ft})) + (3 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))]^{\,2}}{(6.75 \text{ ft})^2 \times [(3 \times (0.0036624 \text{ kipft/ft})) + (2 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$ $$p = -0.00040666 \text{ kip/ft}^2$$ s - Earth pressure against the pile at distance L_e, $$s = \frac{6\, [(2\, M_o) + (H_o\, L_e)]}{L_e^2}$$ $$s = \frac{6 \times [(2 \times (0.0036624 \text{ kipft/ft})) + ((-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$ $$s = -0.0003093 \text{ kip/ft}^2$$ **Check lateral soil pressure capacity:** p_a - Allowable lateral soil pressure at depth a/2, $$p_a = R\, \frac{a}{2}$$ $$p_a = (150 \text{ psf/ft}) \times \frac{(4.8588 \text{ ft})}{2}$$ $$p_a = 0.36441 \text{ kip/ft}^2$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(-0.00040666 \text{ kip/ft}^2\right)}{\left(0.36441 \text{ kip/ft}^2\right)}$$		

$$Ratio = -0.0011159$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

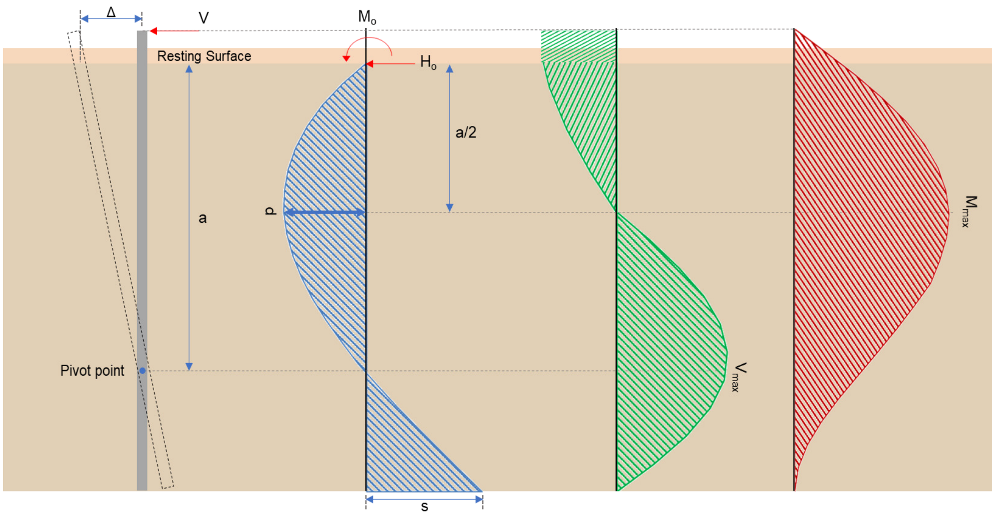
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0003093\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.00030548$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.439\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.70685\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.475\ \mathrm{kipft}) + ((-4.439\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.3559\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.3559\ \mathrm{kipft/ft})}{(-0.70685\ \mathrm{kip/ft})}$$

$$E = 11.821\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3559\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.70685\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{\ldots}$$

$$a = \frac{}{(6 \times (8.3559\ \mathrm{kipft/ft})) + (4 \times (-0.70685\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6551\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.70685\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.821\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6551\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.821\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6551\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.627\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.70685\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.75\ \mathrm{ft})) \times \left[\left(\frac{(11.821\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + \frac{(4.6551\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (11.821\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6551\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.821\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6551\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 34.133\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.015\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.0023885\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.039\ \mathrm{kipft}) + ((-0.015\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.0062102\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0062102\ \mathrm{kipft/ft})}{(-0.0023885\ \mathrm{kip/ft})}$$

$$E = 2.6\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0062102\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.0023885\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.0062102\ \mathrm{kipft/ft})) + (4 \times (-0.0023885\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8565\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.0023885\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.8565\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.6\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.8565\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.012903\ \text{kip}$$

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

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

$$M_{max} = ((-0.0023885\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(2.6\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.8565\ \text{ft})}{2 \times (6.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (2.6\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.8565\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.6\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.8565\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.037816\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(9.016\ \text{kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.297\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.297\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

Status: PASS
Ratio: 0.970

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10⌀: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.016\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0033702$$

Status: PASS
Ratio: 0.000

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

$$d = 0.80 \times (48\ \text{in})$$

$$d = 38.4\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.016 \text{ kip} \rightarrow 9016 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9016 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.69 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.69 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.69 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.69 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.88 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.627 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.627 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.095844$$

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

Considering z-direction:

$V_{max} = 0.012903 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.012903 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.00011637$$

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:
$\phi M_{n,1}$

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

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

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

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

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.133 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(34.133 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13675$$

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

Considering z-direction:

$M_{max} = 0.037816 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(0.037816 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.00015151$$	Status: **PASS** Ratio: **0.000**

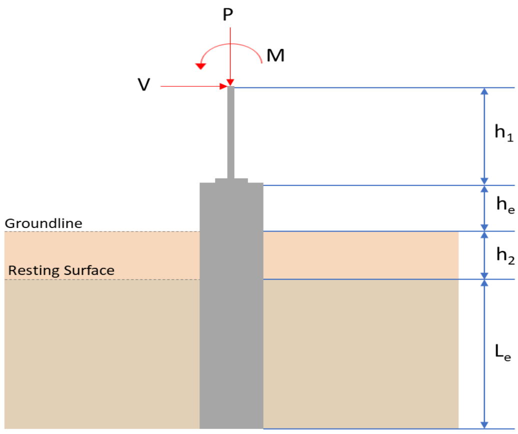
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 6.75$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ 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)	6.247	9.381
V_x (kip)	-2.722	-4.542
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	31.682	53.024

Material Properties

$f'_{ck} = 2.5$ 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,

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

$$H_o = \frac{(-2.722 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43344 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.682\ \mathrm{kipft}) + ((-2.722\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.0449\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.2284\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2284\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 6.228\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.228\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.92267$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.247\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.39044\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39044\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.19522$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.43344\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.0449\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0449\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.43344\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (5.0449\ \mathrm{kipft/ft})) + (4 \times (-0.43344\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6568\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.0449\ \mathrm{kipft/ft})) + (3 \times (-0.43344\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (5.0449\ \mathrm{kipft/ft})) + (2 \times (-0.43344\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.23054\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (5.0449\ \mathrm{kipft/ft})) + ((-0.43344\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.94342\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6568\ \mathrm{ft})}{2}$$

$$p_a = 0.34926\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23054\ \mathrm{kip/ft^2}\right)}{\left(0.34926\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.66008$$

Status: **PASS**
Ratio: **0.660**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

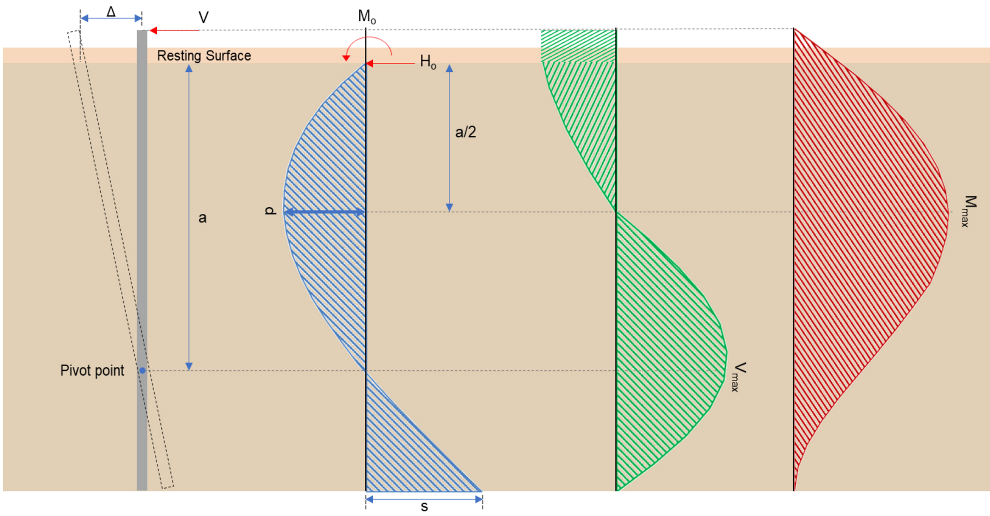
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.94342\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.93177$$

Status: PASS
Ratio: 0.930



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.542\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.72325\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.024\ \text{kipft}) + ((-4.542\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 8.4433\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.4433\ \text{kipft/ft})}{(-0.72325\ \text{kip/ft})}$$

$$E = 11.674\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4433\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.72325\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (8.4433\ \text{kipft/ft})) + (4 \times (-0.72325\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6565\ \text{ft}$$

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

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

$$V_{max} = ((-0.72325\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.674\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6565\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.674\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6565\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.762\ \text{kip}$$

$V_{max} = 10.762$ kip

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

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

$$M_{max} = ((-0.72325\ \mathrm{kip/ft})\times(48\ \mathrm{in})\times(6.75\ \mathrm{ft}))\times\left[\left(\frac{(11.674\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+\frac{(4.6565\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4\times(11.674\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+3\right)\times\left(\frac{(4.6565\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^3\right]+\left[\left(\frac{3\times(11.674\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+2\right)\times\left(\frac{(4.6565\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 34.548\ \mathrm{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.381\ \mathrm{kip})}{(0.65)\times(0.8)}-(0.85\times(2.5\ \mathrm{ksi})\times(2304\ \mathrm{in}^2))}{(60\ \mathrm{ksi})-(0.85\times(2.5\ \mathrm{ksi}))},(0.08\times(2304\ \mathrm{in}^2))\right]$$

$$A_{st,required} = -84.284\ \mathrm{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.284\ \mathrm{in}^2),(0.0018\times(2304\ \mathrm{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10⌀: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.381\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0035067$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (48\ \text{in})$$

$$d = 38.4\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,max} = 296.21\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.381\ \text{kip} \rightarrow 9381\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(9381\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 119.74\ \mathrm{kip}$$

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

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (119.74\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 119.74\ \mathrm{kip}$$

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)

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.74\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 110.91\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 10.762\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.762\ \mathrm{kip})}{(110.91\ \mathrm{kip})}$$

$$Ratio = 0.097032$$

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

14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48\text{ in}) \times (48\text{ in})^2}{6}$$ $$S_m = 18432\text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\text{ ksi}) \times \left(18432\text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6\text{ kipft}$$ 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:** $M_{max} = 34.548\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(34.548\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.13841$$	Status: **PASS** Ratio: **0.140**

REFERENCES	CALCULATIONS	RESULTS

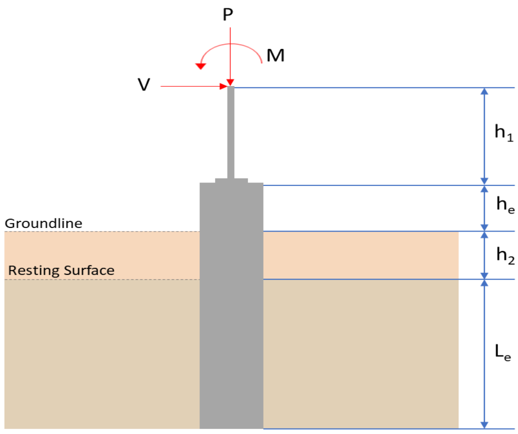
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\text{ in}$ - Pile width
$D = 48\text{ in}$ - Pile depth
$L = 6.75\text{ ft}$ - Total pile length
$h_1 = 0\text{ ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\text{ ft}$ - Depth to resisting surface
$h_e = 0\text{ ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.022	9.016
V_x (kip)	-2.662	-4.439
V_z (kip)	0.009	0.015
M_x (kipft)	-0.023	-0.039
M_z (kipft)	31.258	52.476

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,

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

$$H_o = \frac{(-2.662\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.42389\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.258 \text{ kipft}) + ((-2.662 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9774 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.2154 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.009 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0014331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.023 \text{ kipft}) + ((0.009 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0036624 \text{ kipft/ft}$$

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:
$L_{e,z} = 0.70705 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2154 \text{ ft}), (0.70705 \text{ ft})]$$

$$L_{e,req} = 6.215 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.215 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.92074$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.022\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.37637\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.37637\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.18819$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.42389\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.9774\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9774\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.42389\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (4.9774\ \mathrm{kipft/ft})) + (4 \times (-0.42389\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6558\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9774\ \mathrm{kipft/ft})) + (3 \times (-0.42389\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (4.9774\ \mathrm{kipft/ft})) + (2 \times (-0.42389\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.22927\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.9774\ \mathrm{kipft/ft})) + ((-0.42389\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.93413\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6558\ \mathrm{ft})}{2}$$

$$p_a = 0.34919\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22927\ \mathrm{kip/ft^2}\right)}{\left(0.34919\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65659$$

Status: **PASS**
Ratio: **0.660**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.93413\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9226$$

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

Considering z-direction:

$H_o = 0.0014331\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.0036624\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0036624\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (0.0014331\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.0036624\ \mathrm{kipft/ft})) + (4 \times (0.0014331\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8588\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0036624\ \mathrm{kipft/ft})) + (3 \times (0.0014331\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (0.0036624\ \mathrm{kipft/ft})) + (2 \times (0.0014331\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.0010349\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.0036624\ \mathrm{kipft/ft})) + ((0.0014331\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.0022385\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.8588\ \mathrm{ft})}{2}$$

$$p_a = 0.36441\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0010349\ \mathrm{kip/ft^2}\right)}{\left(0.36441\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0028399$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

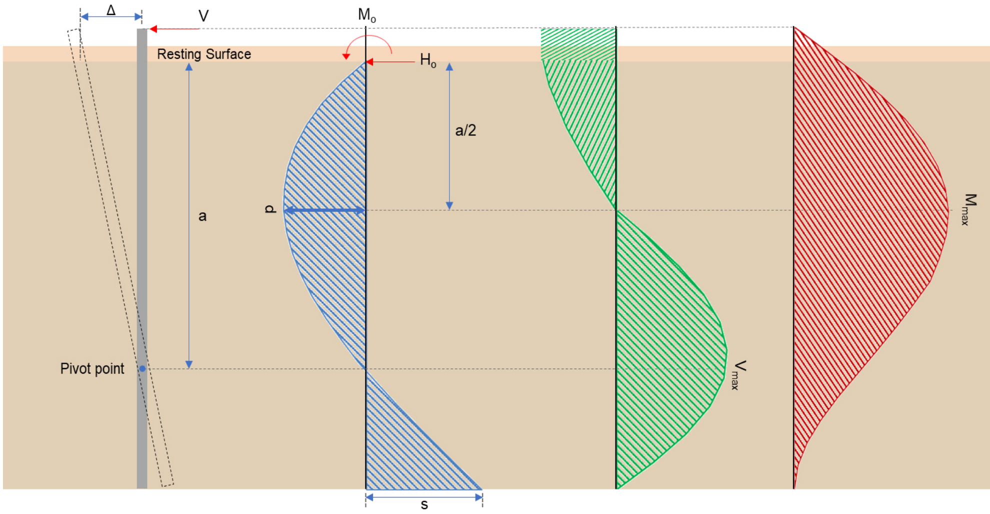
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0022385 \text{ kip/ft}^2\right)}{\left(1.0125 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.0022108$$

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



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.439 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70685 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.476 \text{ kipft}) + ((-4.439 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3561 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.3561 \text{ kipft/ft})}{(-0.70685 \text{ kip/ft})}$$

$$E = 11.822 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3561 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.70685 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.3561\ \text{kipft/ft})) + (4 \times (-0.70685\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6551\ \text{ft}$$

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

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

$$V_{max} = ((-0.70685\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.822\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6551\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.822\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6551\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.627\ \text{kip}$$

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

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

$$M_{max} = ((-0.70685\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(11.822\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.6551\ \text{ft})}{2 \times (6.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (11.822\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6551\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.822\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6551\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 34.133\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.015\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.0023885\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.039\ \text{kipft}) + ((0.015\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.0062102\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0062102\ \text{kipft/ft})}{(0.0023885\ \text{kip/ft})}$$

$$E = 2.6\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0062102\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (0.0023885\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (0.0062102\ \text{kipft/ft})) + (4 \times (0.0023885\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.8565\ \text{ft}$$

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

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

$$V_{max} = ((0.0023885 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8565 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.6 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8565 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.012903 \text{ kip}$$

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

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

$$M_{max} = ((0.0023885 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(2.6 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8565 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (2.6 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8565 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.6 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8565 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.037816 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.016 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.297 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.297 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

Status: PASS
Ratio: 0.970

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.016\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0033702$$

Status: PASS
Ratio: 0.000

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

$$d = 0.80 \times (48\ \text{in})$$

$$d = 38.4\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,max} = 296.21\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.016\ \text{kip} \rightarrow 9016\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(9016\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.69\ \text{kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + (0.05 \times (2500\ \text{psi}))\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,b} = 348.89\ \text{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.69\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.69\ \text{kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.69\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 110.88\ \text{kip}$$

Considering x-direction:

$V_{max} = 10.627\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.627 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.095845$$

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

Considering z-direction:

$V_{max} = 0.012903 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.012903 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.00011637$$

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

14.5.2.1b

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b\, D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

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

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

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

$\phi M_{n,2}$

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

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

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

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:

$M_{max} = 34.133 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(34.133 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13675$$

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

Considering z-direction:

$M_{max} = 0.037816 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(0.037816\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.00015151$$	Status: **PASS** Ratio: **0.000**