Your Project Calculations

Project Name: W12185 - REVISION CU1

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=W12185%20-%20REVISION%20CU1&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/12_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=5rnJrm4cN0GhtoE44LwMCacCsnYmOcrJBhNtwRtvaE0fLylPwtycpy1TqMEi53Fm

Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-XD-57-L-5Hx10W-46J3
Duty Classification:	XD
Module Width:	40.90 in
Module Length:	69.13in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	15
Front Edge Clearance:	10
Total Array Height at Tilt:	14.44 ft
Total Frame Length:	56.50 ft
Frame Weight:	2870 lbs
Array Dimensions N/S:	17.25 ft
Array Dimensions E/W:	58.44 ft
Rail Length:	207.00 in
Rail Spacing:	2.88 ft
Rail Check:	Not Checked

Support Specifications

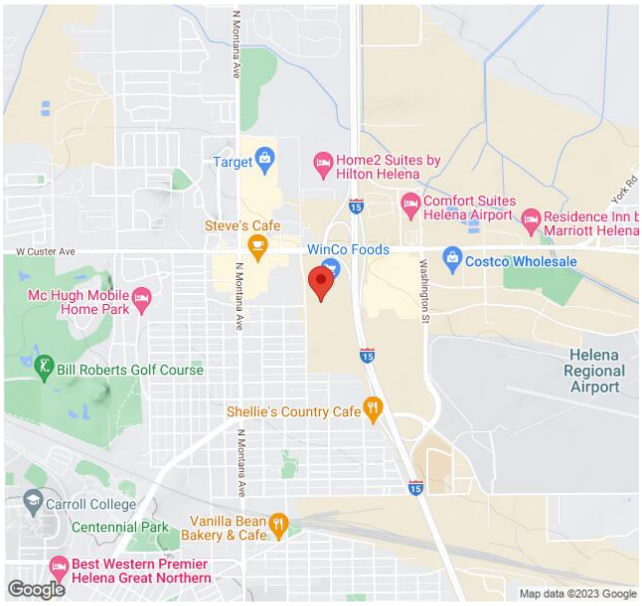
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.23 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.00 ft Pile 3: 6.00 ft
Foundation Volume:	10.667 y^3
Foundation Result:	PASSED
Mount Twist:	0.046850 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2845 N Sanders St, Helena, MT 59601, USA
Wind Speed:	106 mph
Snow Load:	30 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.018144 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

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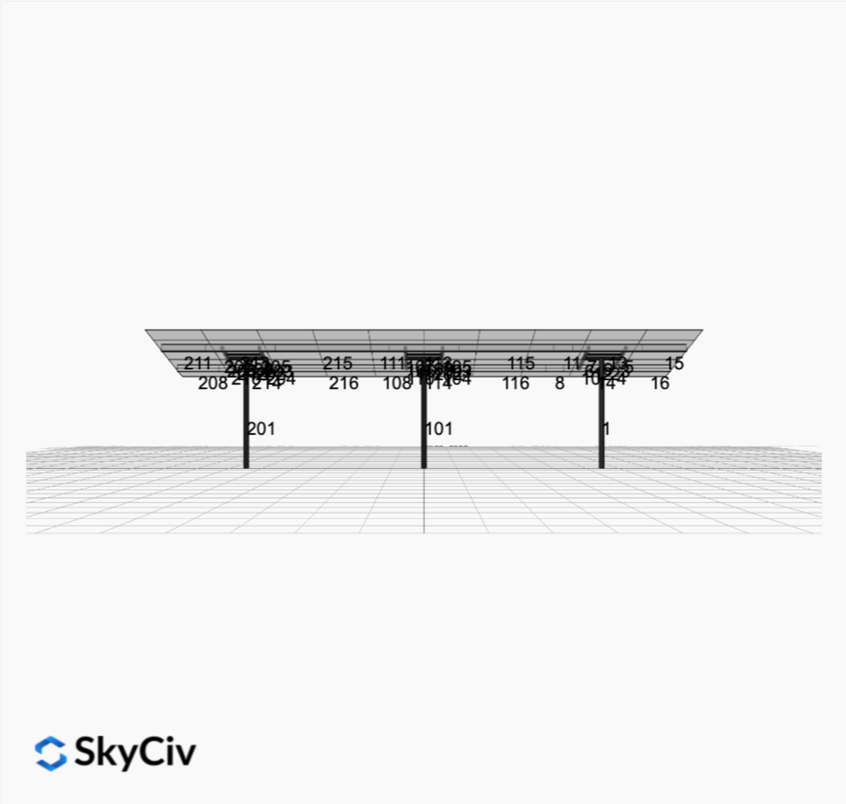
Design Notes:

- 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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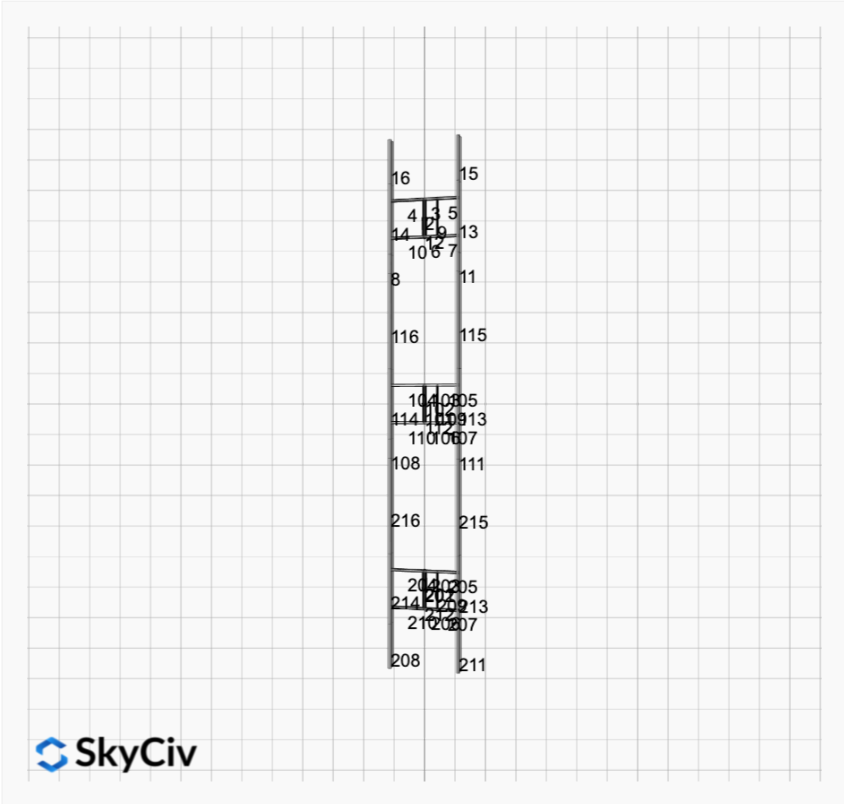
201
101
SkyCiv


201
8.23
SkyCiv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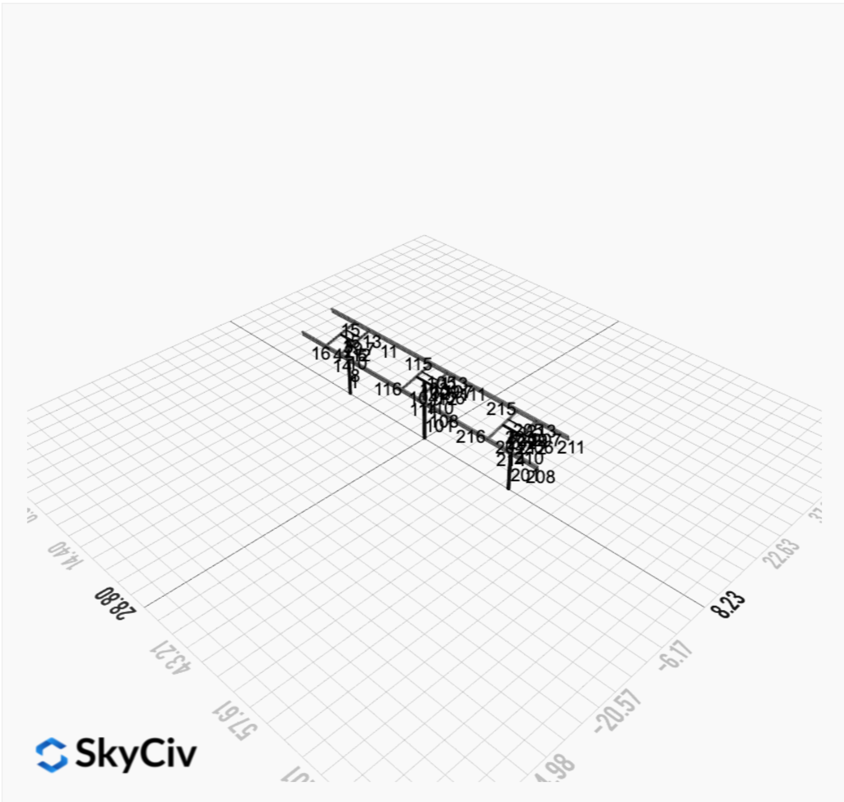
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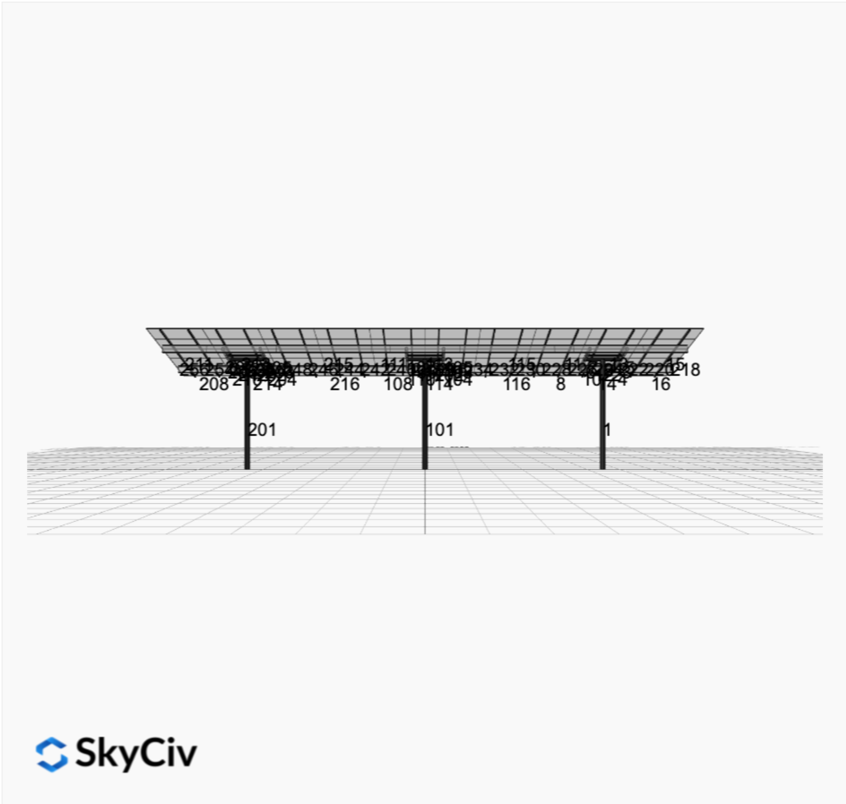
8.23
-6.17
-20.57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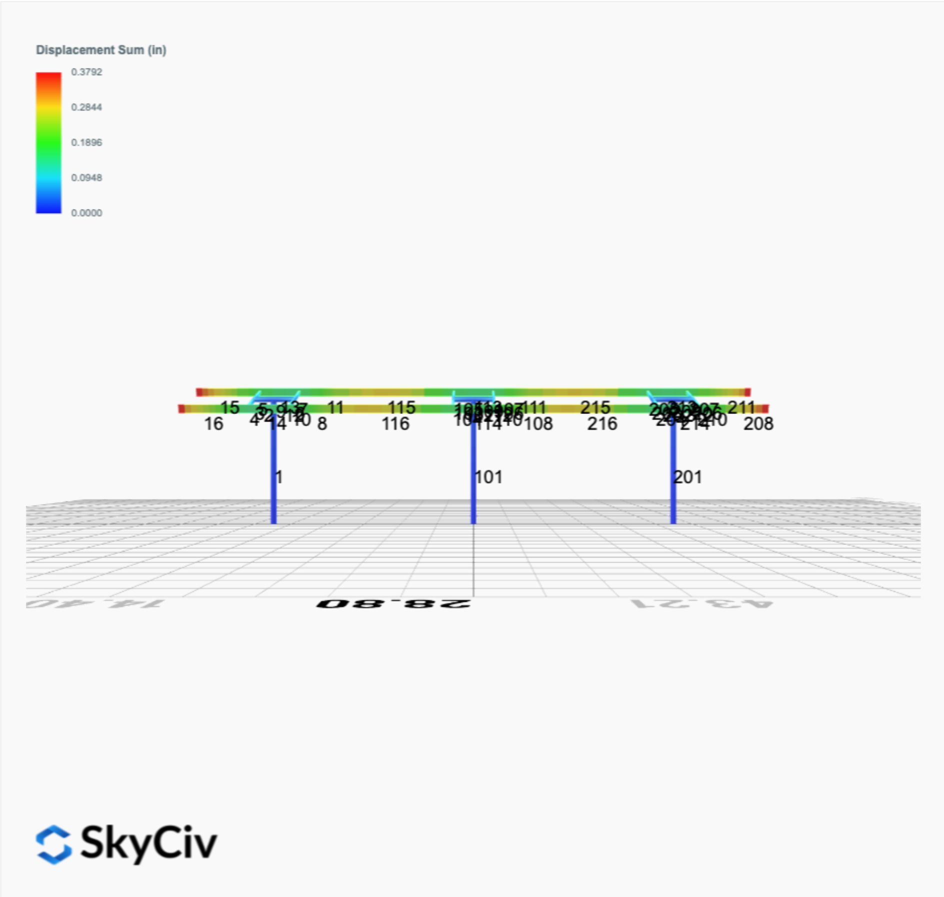
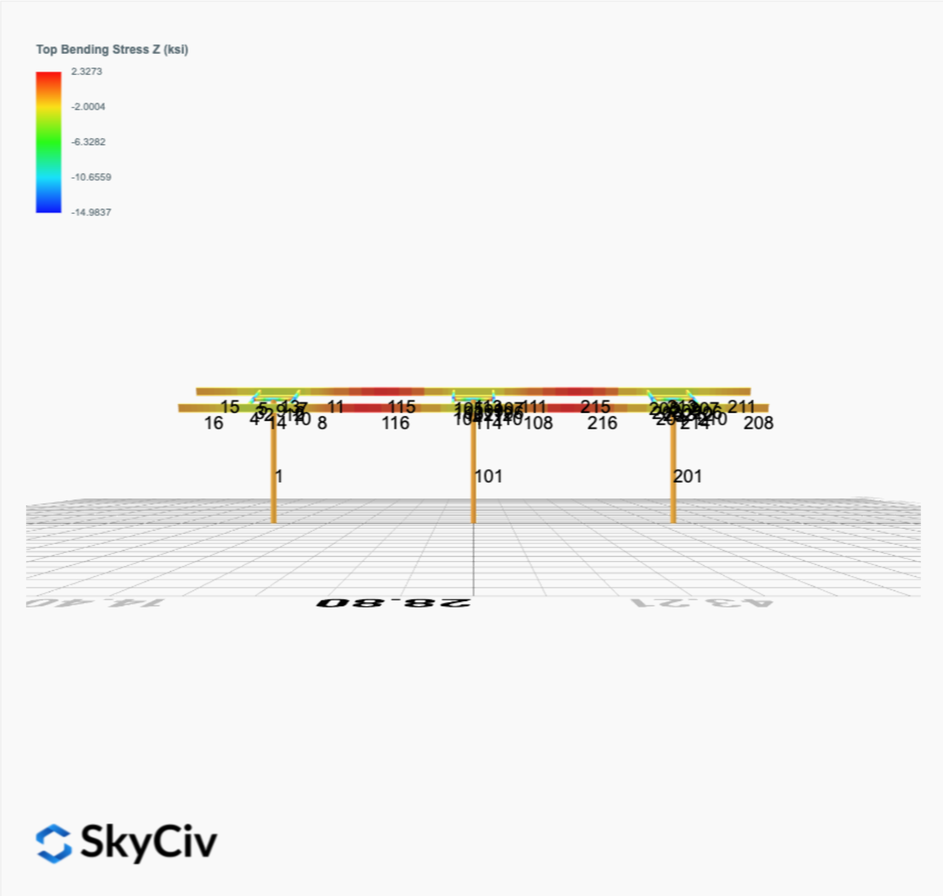
108
116
8
16
201
101
SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
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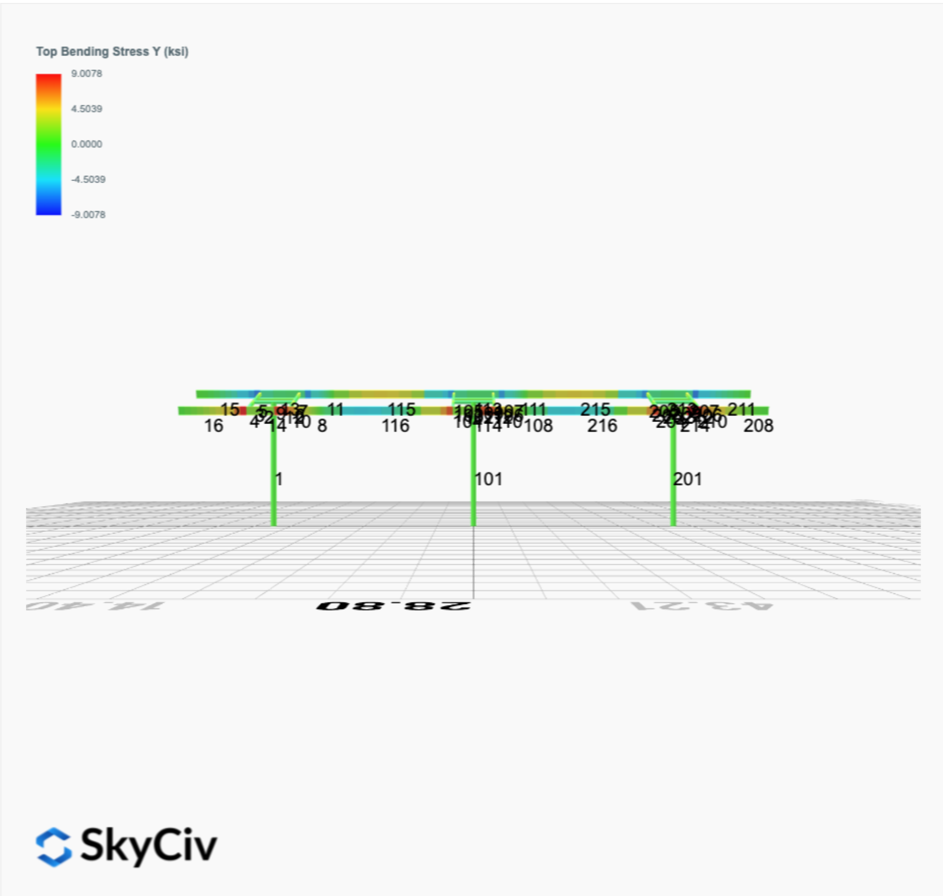
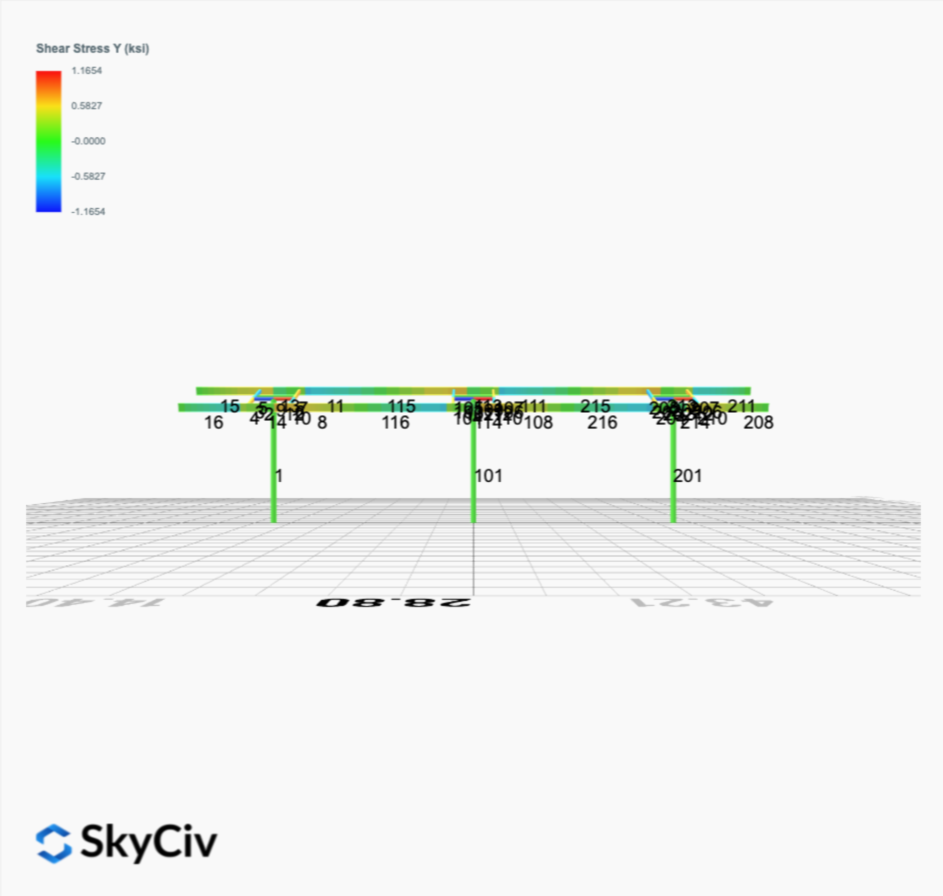
-2.0004
-6.3282
-10.6559
-14.9837
SkyCiv


Top Bending Stress Y (ksi)
9.0078
4.5039
0.0000
-4.5039
-9.0078
SkyCiv


Shear Stress Y (ksi)
1.1654
0.5827
-0.0000
-0.5827
-1.1654
15
16
11
8
115
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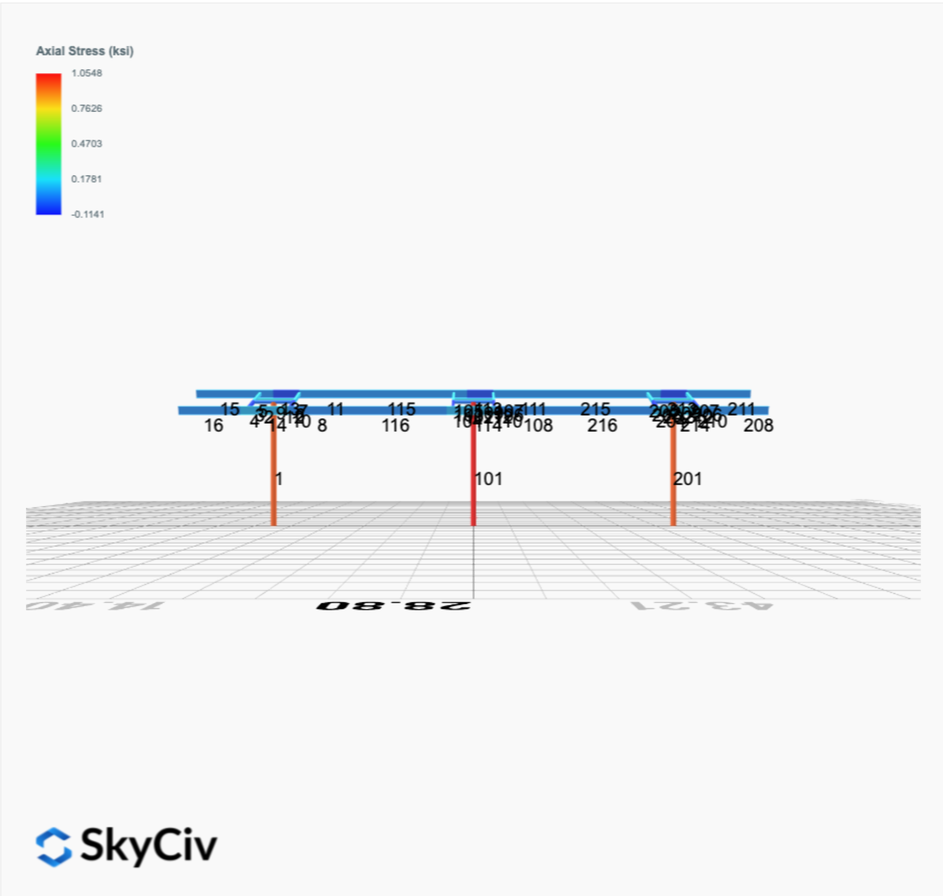
208
1
101
201
28.80
SkyCiv


Axial Stress (ksi)
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0.1781
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16
11
8
115
116
108
215
216
211
208
1
101
201
28.80
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	-0.0028	2.4671	-0.0036	-0.0139	0.0076	0.0581
ULS: 2. D + L	-0.0028	2.4671	-0.0036	-0.0139	0.0076	0.0581
ULS: 3. D + (S or Lr or R)	-0.0104	8.0640	-0.0137	-0.0525	0.0290	0.1476
ULS: 3. D + (S or Lr or R)	-0.0028	2.4671	-0.0036	-0.0139	0.0076	0.0581
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0085	6.6648	-0.0112	-0.0429	0.0236	0.1252
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0028	2.4671	-0.0036	-0.0139	0.0076	0.0581
ULS: 5b. D + 0.7E	-0.0028	2.4671	-0.0036	-0.0139	0.0076	0.0581
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0085	6.6648	-0.0112	-0.0429	0.0236	0.1252
ULS: 8. 0.6D + 0.7E	-0.0017	1.4803	-0.0022	-0.0084	0.0045	0.0349
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1545	6.7396	-0.0061	-0.0238	0.0072	17.0457
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.1545	6.7396	-0.0061	-0.0238	0.0072	17.0457
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8682	-0.7718	-0.0002	-0.0005	0.0061	-7.9930
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7557	-0.3350	-0.0041	-0.0155	0.0102	-19.1626
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8723	9.8691	-0.0131	-0.0503	0.0234	12.8659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8723	9.8691	-0.0131	-0.0503	0.0234	12.8659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6447	4.2356	-0.0087	-0.0328	0.0225	-5.9132
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5603	4.5632	-0.0115	-0.0440	0.0256	-14.2903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8666	5.6715	-0.0055	-0.0213	0.0073	12.7988
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8666	5.6715	-0.0055	-0.0213	0.0073	12.7988
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6505	0.0380	-0.0011	-0.0039	0.0065	-5.9802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5661	0.3655	-0.0040	-0.0151	0.0095	-14.3574
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1534	5.7527	-0.0046	-0.0182	0.0042	17.0225
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.1534	5.7527	-0.0046	-0.0182	0.0042	17.0225
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8693	-1.7586	0.0012	0.0050	0.0030	-8.0163
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7568	-1.3218	-0.0026	-0.0099	0.0071	-19.1858

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	15.4766
Shear X	-1.9281
Shear Z	-0.0223
Moment X	-0.0864
Moment Y (Twist)	0.0470
Moment Z	34.0506

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	9.8691
Shear X	-1.1545
Shear Z	-0.0137
Moment X	-0.0525
Moment Y (Twist)	0.0290
Moment Z	19.1858

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	0.0055	2.5683	-0.0000	0.0000	0.0000	-0.0177
ULS: 2. D + L	0.0055	2.5683	-0.0000	0.0000	0.0000	-0.0177
ULS: 3. D + (S or Lr or R)	0.0208	8.4556	0.0000	0.0000	-0.0000	-0.1433
ULS: 3. D + (S or Lr or R)	0.0055	2.5683	-0.0000	0.0000	0.0000	-0.0177
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0170	6.9838	0.0000	0.0000	-0.0000	-0.1119
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0055	2.5683	-0.0000	0.0000	0.0000	-0.0177
ULS: 5b. D + 0.7E	0.0055	2.5683	-0.0000	0.0000	0.0000	-0.0177

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0170	6.9838	0.0000	0.0000	-0.0000	-0.1119
ULS: 8. 0.6D + 0.7E	0.0033	1.5410	-0.0000	0.0000	0.0000	-0.0106
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1749	7.0259	-0.0000	0.0000	0.0000	17.3211
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.1749	7.0259	-0.0000	0.0000	0.0000	17.3211
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.9067	-0.8179	-0.0000	0.0000	0.0000	-8.2841
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7712	-0.3463	-0.0000	0.0000	0.0000	-19.4990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8683	10.3270	0.0000	0.0000	-0.0000	12.8922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8683	10.3270	0.0000	0.0000	-0.0000	12.8922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6929	4.4442	0.0000	0.0000	-0.0000	-6.3117
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5913	4.7978	0.0000	0.0000	-0.0000	-14.7228
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8798	5.9115	-0.0000	0.0000	0.0000	12.9864
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8798	5.9115	-0.0000	0.0000	0.0000	12.9864
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6814	0.0287	-0.0000	0.0000	0.0000	-6.2175
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5798	0.3823	-0.0000	0.0000	0.0000	-14.6286
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1771	5.9986	-0.0000	0.0000	0.0000	17.3282
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.1771	5.9986	-0.0000	0.0000	0.0000	17.3282
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.9045	-1.8452	-0.0000	0.0000	0.0000	-8.2770
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7690	-1.3736	-0.0000	0.0000	0.0000	-19.4919

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	16.2149
Shear X	-1.9674
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	34.7263

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	10.3270
Shear X	-1.1771
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	19.4990

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	-0.0028	2.4671	0.0036	0.0139	-0.0076	0.0581
ULS: 2. D + L	-0.0028	2.4671	0.0036	0.0139	-0.0076	0.0581
ULS: 3. D + (S or Lr or R)	-0.0104	8.0640	0.0137	0.0525	-0.0290	0.1476
ULS: 3. D + (S or Lr or R)	-0.0028	2.4671	0.0036	0.0139	-0.0076	0.0581
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0085	6.6648	0.0112	0.0429	-0.0236	0.1252
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0028	2.4671	0.0036	0.0139	-0.0076	0.0581
ULS: 5b. D + 0.7E	-0.0028	2.4671	0.0036	0.0139	-0.0076	0.0581
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0085	6.6648	0.0112	0.0429	-0.0236	0.1252
ULS: 8. 0.6D + 0.7E	-0.0017	1.4803	0.0022	0.0084	-0.0045	0.0349
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1545	6.7396	0.0061	0.0238	-0.0072	17.0457
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.1545	6.7396	0.0061	0.0238	-0.0072	17.0457
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8682	-0.7718	0.0002	0.0005	-0.0061	-7.9930
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7557	-0.3350	0.0041	0.0155	-0.0102	-19.1626
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8723	9.8691	0.0131	0.0503	-0.0234	12.8659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8723	9.8691	0.0131	0.0503	-0.0234	12.8659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6447	4.2356	0.0087	0.0328	-0.0225	-5.9132
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5603	4.5632	0.0115	0.0440	-0.0256	-14.2903

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	-0.8666	5.6715	0.0055	0.0213	-0.0073	12.7988
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8666	5.6715	0.0055	0.0213	-0.0073	12.7988
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6505	0.0380	0.0011	0.0039	-0.0065	-5.9802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5661	0.3655	0.0040	0.0151	-0.0095	-14.3574
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1534	5.7527	0.0046	0.0182	-0.0042	17.0225
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.1534	5.7527	0.0046	0.0182	-0.0042	17.0225
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8693	-1.7586	-0.0012	-0.0050	-0.0030	-8.0163
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7568	-1.3218	0.0026	0.0099	-0.0071	-19.1858

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	15.4766
Shear X	-1.9281
Shear Z	0.0223
Moment X	0.0865
Moment Y (Twist)	0.0469
Moment Z	34.0517

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	9.8691
Shear X	-1.1545
Shear Z	0.0137
Moment X	0.0525
Moment Y (Twist)	0.0290
Moment Z	19.1858

Project Details

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

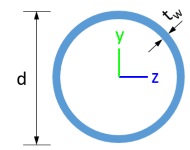
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

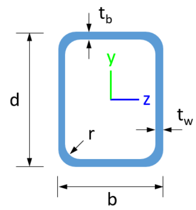
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

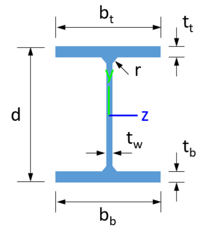
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
7	6in Pipe Sch 40	6.63	0.28					



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



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	25.69	25.69	12.23	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.43,1.17,1.18,1.18,1.17,1.18	300	200	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.73,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.46,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.49,1.17,1.18,1.18,1.17,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.74,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.98,1.66,1.67,1.67,1.66,1.66	300	200	1
8	20	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.27,2.11,2.11,2.12,2.37,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.28,2.11,2.11,2.12,2.34	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
11	20	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.29,2.24,2.12,2.12,2.12,2.18,2.12,2.12,2.12,2.11,2.12,2.12,2.16,2.24,2.12,2.12,2.12,2.17	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.34,1.12,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.09,1.09,1.09,1.00,1.12,1.09,1.09,1.09,1.10	300	200	1
14	20	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.11,1.11,1.09,1.09,1.09,1.25,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.12,1.09,1.09,1.09,1.35	300	200	1
15	20	9.97	9.97	4.75	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	20	9.97	9.97	4.75	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	7	25.69	25.69	12.23	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.22,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.39,1.17,1.18,1.18,1.17,1.18	300	200	1
104	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.72,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.19,1.66,1.67,1.67,1.66,1.66	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.22,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.39,1.17,1.18,1.18,1.17,1.18	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.72,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.19,1.66,1.67,1.67,1.66,1.66	300	200	1

108	20	1.33	1.33	2.05	2.06,2.06,2.06,2.06,2.06,2.06,2.06,2.06,2.06,1.94,2.06,2.06,2.06,1.76,2.06,2.06,2.06,2.06,2.06,2.06,2.06,1.91,2.06,2.06,2.06,1.68	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
111	20	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.04,2.04,1.58,1.61,1.99,1.99,1.83,1.78,2.06,2.06,2.08,2.12,2.05,2.05,1.06,1.64,1.97,1.97,1.90,1.81	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1
113	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.11,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.03,1.04,1.04,1.45,1.10,1.04,1.04,1.04,1.05	300	200	1
114	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.07,1.04,1.04,1.04,1.33,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.07,1.04,1.04,1.04,1.62	300	200	1
115	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.83,1.11,1.13,1.13,1.13,1.12,1.14,1.14,1.15,1.17,1.14,1.14,1.48,1.11,1.13,1.13,1.13,1.12	300	200	1
116	20	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.14,1.14,1.14,1.12,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.14,1.14,1.14,1.12	300	200	1
201	7	25.69	25.69	12.23	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.49,1.17,1.18,1.18,1.17,1.18	300	200	1
204	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.74,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.98,1.66,1.67,1.67,1.66,1.66	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.44,1.17,1.18,1.18,1.17,1.18	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.73,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,2.46,1.65,1.67,1.67,1.66,1.66	300	200	1
208	20	9.97	9.97	4.75	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	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.69	300	200	1
211	20	9.97	9.97	4.75	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	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.35,1.12,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.09,1.09,1.09,1.00,1.12,1.09,1.09,1.09,1.10	300	200	1
214	20	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.11,1.11,1.09,1.09,1.09,1.25,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.12,1.09,1.09,1.09,1.35	300	200	1
215	20	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.19,1.18,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.69,1.18,1.17,1.17,1.17,1.18	300	200	1
216	20	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.19,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.19	300	200	1

Member Design Capacity

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	66.91	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	30.66	6.46	56.26	44.91
14	159.30	116.35	33.32	6.46	56.26	44.91
15	159.30	34.37	46.90	6.46	56.26	44.91
16	159.30	34.37	46.90	6.46	56.26	44.91
101	251.16	66.91	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.57	6.46	56.26	44.91
114	159.30	116.35	31.71	6.46	56.26	44.91
115	159.30	75.13	21.41	6.46	56.26	44.91
116	159.30	75.13	21.52	6.46	56.26	44.91
201	251.16	66.91	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	34.37	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	34.37	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	30.71	6.46	56.26	44.91
214	159.30	116.35	33.32	6.46	56.26	44.91
215	159.30	75.13	22.45	6.46	56.26	44.91
216	159.30	75.13	22.53	6.46	56.26	44.91

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.231	0.805	0.004	0.026	0.000	0.814	#16	0.686	Not Required	Pass
2	0.002	0.487	0.077	0.101	0.013	0.551	#21	0.036	Not Required	Pass
3	0.005	0.709	0.026	0.071	0.002	0.737	#21	0.046	Not Required	Pass
4	0.005	0.671	0.089	0.067	0.019	0.726	#21	0.082	Not Required	Pass
5	0.005	0.440	0.093	0.070	0.024	0.462	#21	0.076	Not Required	Pass
6	0.005	0.704	0.030	0.070	0.003	0.736	#21	0.046	Not Required	Pass
7	0.005	0.437	0.097	0.070	0.025	0.463	#21	0.076	Not Required	Pass
8	0.000	0.045	0.101	0.043	0.011	0.137	#21	0.102	Not Required	Pass
9	0.010	0.074	0.027	0.001	0.000	0.100	#21	0.206	Not Required	Pass
10	0.005	0.665	0.095	0.066	0.021	0.730	#21	0.082	Not Required	Pass
11	0.000	0.047	0.102	0.045	0.011	0.141	#21	0.102	Not Required	Pass
12	0.002	0.480	0.076	0.101	0.013	0.542	#21	0.036	Not Required	Pass
13	0.004	0.275	0.259	0.058	0.014	0.515	#21	0.306	Not Required	Pass
14	0.005	0.266	0.259	0.055	0.014	0.502	#21	0.204	Not Required	Pass
15	0.000	0.097	0.138	0.034	0.008	0.235	#21	Not Required	Not Required	Pass
16	0.000	0.092	0.138	0.032	0.008	0.231	#21	Not Required	Not Required	Pass
101	0.242	0.821	0.000	0.026	0.000	0.846	#13	0.686	Not Required	Pass
102	0.002	0.503	0.077	0.106	0.013	0.566	#21	0.036	Not Required	Pass
103	0.005	0.741	0.033	0.074	0.005	0.776	#21	0.046	Not Required	Pass
104	0.005	0.700	0.087	0.070	0.019	0.761	#21	0.082	Not Required	Pass
105	0.005	0.459	0.090	0.073	0.023	0.483	#21	0.076	Not Required	Pass
106	0.005	0.741	0.033	0.074	0.005	0.776	#21	0.046	Not Required	Pass
107	0.005	0.459	0.090	0.073	0.023	0.483	#21	0.076	Not Required	Pass
108	0.000	0.056	0.093	0.041	0.011	0.121	#21	0.102	Not Required	Pass
109	0.008	0.071	0.025	0.001	0.000	0.099	#21	0.206	Not Required	Pass
110	0.005	0.700	0.087	0.070	0.019	0.761	#21	0.082	Not Required	Pass
111	0.000	0.058	0.095	0.043	0.011	0.124	#21	0.102	Not Required	Pass
112	0.002	0.503	0.078	0.106	0.013	0.566	#21	0.036	Not Required	Pass
113	0.004	0.219	0.245	0.055	0.014	0.440	#21	0.306	Not Required	Pass
114	0.004	0.213	0.243	0.053	0.014	0.428	#21	0.306	Not Required	Pass
115	0.001	0.263	0.134	0.043	0.011	0.398	#21	0.507	Not Required	Pass
116	0.000	0.252	0.135	0.041	0.011	0.387	#21	0.507	Not Required	Pass
201	0.231	0.805	0.004	0.026	0.000	0.814	#16	0.686	Not Required	Pass
202	0.002	0.480	0.076	0.101	0.013	0.542	#21	0.036	Not Required	Pass
203	0.005	0.704	0.030	0.070	0.003	0.736	#21	0.046	Not Required	Pass
204	0.005	0.665	0.095	0.066	0.021	0.730	#21	0.082	Not Required	Pass
205	0.005	0.437	0.097	0.070	0.025	0.463	#21	0.076	Not Required	Pass
206	0.005	0.709	0.026	0.071	0.002	0.737	#21	0.046	Not Required	Pass
207	0.005	0.440	0.093	0.070	0.024	0.462	#21	0.076	Not Required	Pass
208	0.000	0.092	0.138	0.032	0.008	0.231	#21	Not Required	Not Required	Pass
209	0.010	0.074	0.027	0.001	0.000	0.100	#21	0.206	Not Required	Pass
210	0.005	0.671	0.089	0.067	0.019	0.726	#21	0.082	Not Required	Pass
211	0.000	0.097	0.138	0.034	0.008	0.235	#21	Not Required	Not Required	Pass
212	0.002	0.487	0.077	0.101	0.013	0.551	#21	0.036	Not Required	Pass
213	0.004	0.275	0.259	0.058	0.014	0.515	#21	0.204	Not Required	Pass
214	0.005	0.266	0.259	0.055	0.014	0.502	#21	0.306	Not Required	Pass
215	0.001	0.259	0.135	0.045	0.011	0.393	#21	0.507	Not Required	Pass
216	0.000	0.246	0.136	0.043	0.011	0.382	#21	0.507	Not Required	Pass

Definitions

Φ_t Safety factor for tensile

Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comresssion)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

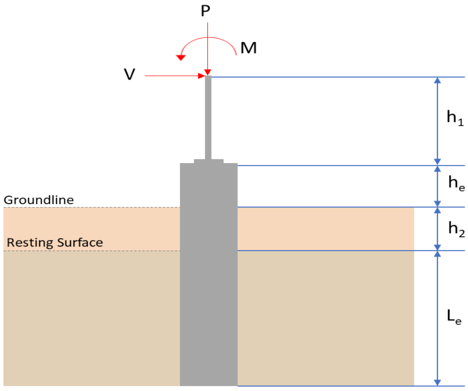
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 6$ 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)	9.869	15.477
V_x (kip)	-1.155	-1.928
V_z (kip)	-0.014	-0.022
M_x (kipft)	-0.053	-0.086
M_z (kipft)	19.186	34.051

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,

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

$$H_o = \frac{(-1.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18392 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.186\ \mathrm{kipft}) + ((-1.155\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 3.0551\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.6658\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.0022293\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.053\ \mathrm{kipft}) + ((-0.014\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.0084395\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 0.82651\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6658\ \mathrm{ft}), (0.82651\ \mathrm{ft})]$$

$$L_{e,req} = 5.666\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.666\ \mathrm{ft})}{(6\ \mathrm{ft})}$$

$$Ratio = 0.94433$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.869\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.61681\ \mathrm{kip/ft}^2$$

$q = 0.61681\ \mathrm{kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.61681\ \mathrm{kip/ft^2})}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.30841$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.18392\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.0551\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0551\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.18392\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (3.0551\ \mathrm{kipft/ft})) + (4 \times (-0.18392\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.097\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0551\ \mathrm{kipft/ft})) + (3 \times (-0.18392\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^2}{(6\ \mathrm{ft})^2 \times [(3 \times (3.0551\ \mathrm{kipft/ft})) + (2 \times (-0.18392\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = 0.23768\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.0551\ \mathrm{kipft/ft})) + ((-0.18392\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.83445\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.097\ \mathrm{ft})}{2}$$

$$p_a = 0.30728\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.23768\ \mathrm{kip/ft^2})}{(0.30728\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.77352$$

Status: **PASS**
Ratio: **0.770**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.83445\ \mathrm{kip/ft^2}\right)}{\left(0.9\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.92716$$

Status: PASS
Ratio: 0.930

Considering z-direction:

$H_o = -0.0022293\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.0084395\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{\left(4 \times (0.0084395\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})\right) + \left(3 \times (-0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{\left(6 \times (0.0084395\ \mathrm{kipft/ft})\right) + \left(4 \times (-0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})\right)}$$

$$a = 4.2569\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times \left[\left(4 \times (0.0084395\ \mathrm{kipft/ft})\right) + \left(3 \times (-0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})\right)\right]^2}{(6\ \mathrm{ft})^2 \times \left[\left(3 \times (0.0084395\ \mathrm{kipft/ft})\right) + \left(2 \times (-0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})\right)\right]}$$

$$p = -0.00058976\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times \left[\left(2 \times (0.0084395\ \mathrm{kipft/ft})\right) + \left(\left(-0.0022293\ \mathrm{kip/ft}\right) \times (6\ \mathrm{ft})\right)\right]}{(6\ \mathrm{ft})^2}$$

$$s = 0.00058386\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2569\ \mathrm{ft})}{2}$$

$$p_a = 0.31927\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.00058976\ \mathrm{kip/ft^2}\right)}{\left(0.31927\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.0018472$$

Status: PASS
Ratio: 0.000

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

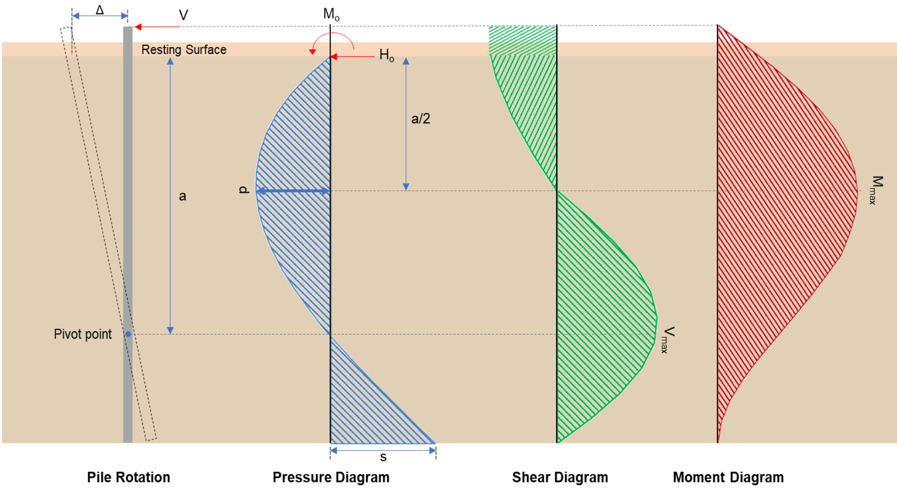
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.00058386\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.00064874$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.928\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.30701\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.051\ \mathrm{kipft}) + ((-1.928\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.4221\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.4221\ \mathrm{kipft/ft})}{(-0.30701\ \mathrm{kip/ft})}$$

$$E = 17.661\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.4221\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.30701\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (5.4221\ \mathrm{kipft/ft})) + (4 \times (-0.30701\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.0923\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.30701\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.661\ \mathrm{ft})}{(6\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.0923\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (17.661\ \mathrm{ft})}{(6\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.0923\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 7.2121\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.30701 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(17.661 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.661 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.661 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.963 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.022 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0035032 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.086 \text{ kipft}) + ((-0.022 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013694 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013694 \text{ kipft/ft})}{(-0.0035032 \text{ kip/ft})}$$

$$E = 3.9091 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013694 \text{ kipft/ft}) \times (6 \text{ ft})) + \left(3 \times (-0.0035032 \text{ kip/ft}) \times (6 \text{ ft})^2\right)}{(6 \times (0.013694 \text{ kipft/ft})) + (4 \times (-0.0035032 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2529 \text{ ft}$$

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

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

$$V_{max} = ((-0.0035032 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2529 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2529 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.025455 \text{ kip}$$

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

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

$$M_{max} = ((-0.0035032 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.9091 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.068838 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(15.477 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.082 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.082 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(15.477\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0057854$$

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

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 $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 15.477\ \text{kip} \rightarrow 15477\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(15477\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 120.55\ \text{kip}$$

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

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

$$V_c = Min\left[(296.21\ \text{kip}), (120.55\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 120.55\ \text{kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.55 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.44 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.2121 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.2121 \text{ kip})}{(111.44 \text{ kip})}$$

$$Ratio = 0.064719$$

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

Considering z-direction:

$V_{max} = 0.025455 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.025455 \text{ kip})}{(111.44 \text{ kip})}$$

$$Ratio = 0.00022842$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 20.963 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(20.963 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.083987$$	Status: **PASS** Ratio: **0.080**
	Considering z-direction: $M_{max} = 0.068838 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.068838 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.00027579$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

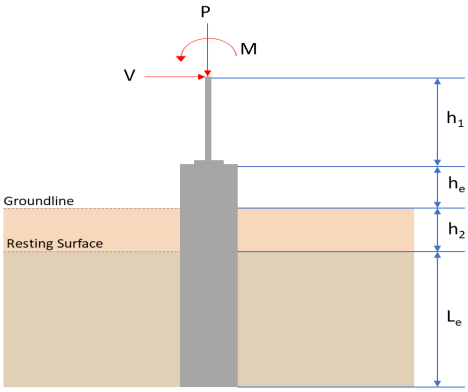
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	10.327	16.215
V_x (kip)	-1.177	-1.967
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	19.499	34.726

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,

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

$$H_o = \frac{(-1.177\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.18742\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.5\ \text{kipft}) + ((-1.177\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.1049\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.6918\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(5.6918\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 5.692\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.692\ \text{ft})}{(6\ \text{ft})}$$

$$Ratio = 0.94867$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(10.327\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.64544\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.64544\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.32272$$

Status: **PASS**
Ratio: **0.320**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.18742\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.1049\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1049\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.18742\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (3.1049\ \mathrm{kipft/ft})) + (4 \times (-0.18742\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.0972\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1049\ \mathrm{kipft/ft})) + (3 \times (-0.18742\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^2}{(6\ \mathrm{ft})^2 \times [(3 \times (3.1049\ \mathrm{kipft/ft})) + (2 \times (-0.18742\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = 0.24128\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.1049\ \mathrm{kipft/ft})) + ((-0.18742\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.84756\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.0972\ \mathrm{ft})}{2}$$

$$p_a = 0.30729\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.24128\ \mathrm{kip/ft^2})}{(0.30729\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.78519$$

Status: **PASS**
Ratio: **0.790**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

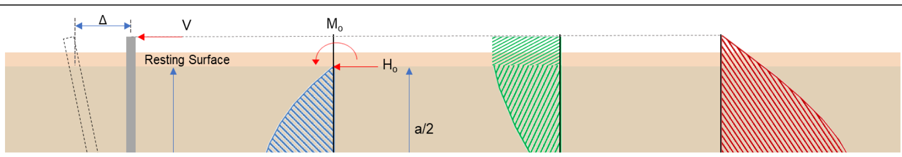
$Ratio$ - Lateral soil capacity

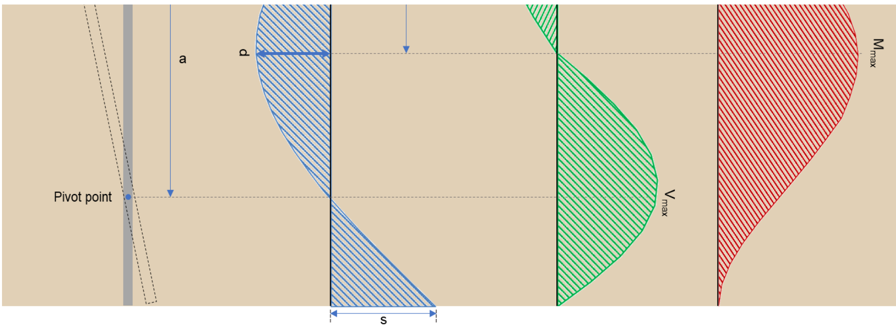
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.84756\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.94173$$

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





Pile Rotation　Pressure Diagram　Shear Diagram　Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.967\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.31322\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.726\text{ kipft}) + ((-1.967\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 5.5296\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5296\text{ kipft/ft})}{(-0.31322\text{ kip/ft})}$$

$$E = 17.654\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5296\text{ kipft/ft}) \times (6\text{ ft})) + \left(3 \times (-0.31322\text{ kip/ft}) \times (6\text{ ft})^2\right)}{(6 \times (5.5296\text{ kipft/ft})) + (4 \times (-0.31322\text{ kip/ft}) \times (6\text{ ft}))}$$

$$a = 4.0924\text{ ft}$$

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

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

$$V_{max} = ((-0.31322\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.654\text{ ft})}{(6\text{ ft})} + 3\right) \times \left(\frac{(4.0924\text{ ft})}{(6\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (17.654\text{ ft})}{(6\text{ ft})} + 2\right) \times \left(\frac{(4.0924\text{ ft})}{(6\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 7.3554\text{ kip}$$

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

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

$$M_{max} = ((-0.31322\text{ kip/ft}) \times (48\text{ in}) \times (6\text{ ft})) \times \left[\left(\frac{(17.654\text{ ft})}{(6\text{ ft})} + \frac{(4.0924\text{ ft})}{2 \times (6\text{ ft})}\right) - \left[\left(\frac{4 \times (17.654\text{ ft})}{(6\text{ ft})} + 3\right) \times \left(\frac{(4.0924\text{ ft})}{2 \times (6\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (17.654\text{ ft})}{(6\text{ ft})} + 2\right) \times \left(\frac{(4.0924\text{ ft})}{2 \times (6\text{ ft})}\right)^4\right]\right]$$

	$M_{max} = 21.379 \text{ kipft}$		

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(16.215 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.057 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.057 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

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

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

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength	
	$$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$	
	$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$	
	$$\phi P_N = 2675.2\ \mathrm{kip}$$	
	$Ratio$ - Capacity	
	$$Ratio = \frac{P}{\phi P_N}$$	
	$$Ratio = \frac{(16.215\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$	
	$$Ratio = 0.0060613$$	Status: **PASS** Ratio: **0.010**

	Shear Strength (ACI 318-19, LRFD)	
	Parameters:	
	$b_w = 48\ \mathrm{in}$ - Effective width,	
22.5.2.2	d - Effective depth	
	$$d = 0.80\, D$$	
	$$d = 0.80 \times (48\ \mathrm{in})$$	
	$$d = 38.4\ \mathrm{in}$$	
22.5.5.1.3	λ_s - size effect modification factor	
	$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$	
	$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\ \mathrm{in})}{10}}}, 1\right]$$	
	$$\lambda_s = 0.64282$$	
	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete	
	$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$	
	$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$	
	$$V_{c,max} = 296.21\ \mathrm{kip}$$	
	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 16.215\ \mathrm{kip} \rightarrow 16215\ \mathrm{lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a)	
	$$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right] b_w\, d$$	
	$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(16215\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$	
	$$V_{c,a} = 120.65\ \mathrm{kip}$$	
	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	
	$$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$	
	$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.65\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$	
	$$V_c = 120.65\ \mathrm{kip}$$	

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\ b_w\ d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{s,a} = 737.28\ \text{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\ d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$ $$A_v = 0.11045\ \text{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\ A_v\ f_{ywk}\ d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times (0.11045\ \text{in}^2) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$ $$V_{s,b} = 50.894\ \text{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$ $$V_s = 50.894\ \text{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.65\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 111.5\ \text{kip}$$ **Considering x-direction:** $V_{max} = 7.3554\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(7.3554\ \text{kip})}{(111.5\ \text{kip})}$$ $$Ratio = 0.065967$$	Status: **PASS** Ratio: **0.070**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\ D^2}{6}$$ $$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$ $$S_m = 18432\ \text{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f'_c\ S_m$$	

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 21.379\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(21.379\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$

$$Ratio = 0.085655$$

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

REFERENCES	CALCULATIONS	RESULTS

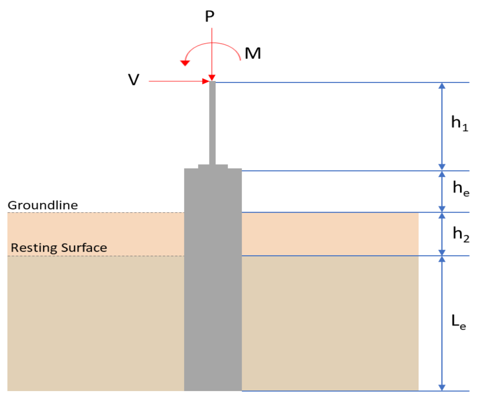
SkyCiv Foundation Design

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
$L = 6 \text{ ft}$ - 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)	9.869	15.477
V_x (kip)	-1.155	-1.928
V_z (kip)	0.014	0.022
M_x (kipft)	0.053	0.086
M_z (kipft)	19.186	34.052

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,

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

$$H_o = \frac{(-1.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18392 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.186\ \mathrm{kipft}) + ((-1.155\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 3.0551\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.6658\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.014\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.0022293\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.053\ \mathrm{kipft}) + ((0.014\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.0084395\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 0.92808\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6658\ \mathrm{ft}), (0.92808\ \mathrm{ft})]$$

$$L_{e,req} = 5.666\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.666\ \mathrm{ft})}{(6\ \mathrm{ft})}$$

$$Ratio = 0.94433$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.869\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.61681\ \mathrm{kip/ft}^2$$

$q = 0.61681\ \text{kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.61681\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.30841$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.18392\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.0551\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0551\ \text{kipft/ft}) \times (6\ \text{ft})) + \left(3 \times (-0.18392\ \text{kip/ft}) \times (6\ \text{ft})^2\right)}{(6 \times (3.0551\ \text{kipft/ft})) + (4 \times (-0.18392\ \text{kip/ft}) \times (6\ \text{ft}))}$$

$$a = 4.097\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0551\ \text{kipft/ft})) + (3 \times (-0.18392\ \text{kip/ft}) \times (6\ \text{ft}))]^{\ 2}}{(6\ \text{ft})^2 \times [(3 \times (3.0551\ \text{kipft/ft})) + (2 \times (-0.18392\ \text{kip/ft}) \times (6\ \text{ft}))]}$$

$$p = 0.23768\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.0551\ \text{kipft/ft})) + ((-0.18392\ \text{kip/ft}) \times (6\ \text{ft}))]}{(6\ \text{ft})^2}$$

$$s = 0.83445\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.097\ \text{ft})}{2}$$

$$p_a = 0.30728\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23768\ \text{kip/ft}^2\right)}{\left(0.30728\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.77352$$

Status: **PASS**
Ratio: **0.770**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.83445\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.92716$$

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

Considering z-direction:

$H_o = 0.0022293\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.0084395\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0084395\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (0.0084395\ \mathrm{kipft/ft})) + (4 \times (0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.2569\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0084395\ \mathrm{kipft/ft})) + (3 \times (0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^{\,2}}{(6\ \mathrm{ft})^2 \times [(3 \times (0.0084395\ \mathrm{kipft/ft})) + (2 \times (0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = 0.0021842\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.0084395\ \mathrm{kipft/ft})) + ((0.0022293\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.0050425\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2569\ \mathrm{ft})}{2}$$

$$p_a = 0.31927\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0021842\ \mathrm{kip/ft^2})}{(0.31927\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.0068412$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

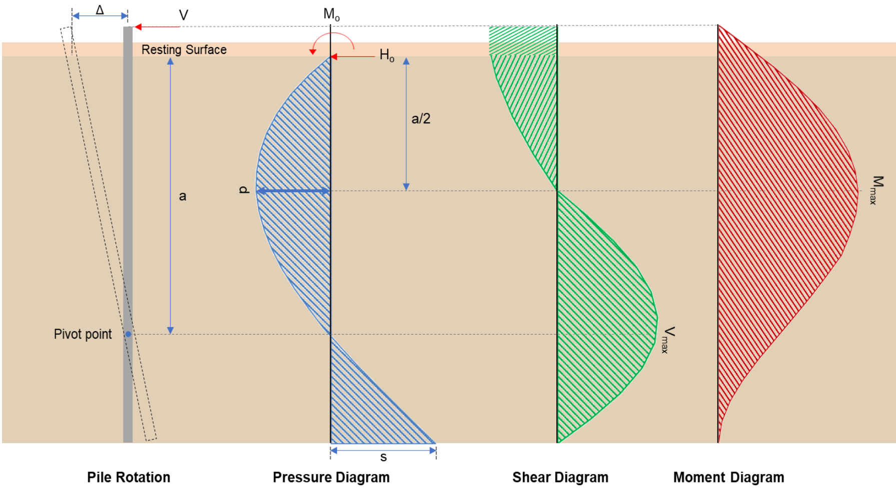
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0050425\ \mathrm{kip/ft^2}\right)}{\left(0.9\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0056027$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.928\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.30701\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.052\ \mathrm{kipft}) + ((-1.928\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.4223\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.4223\ \mathrm{kipft/ft})}{(-0.30701\ \mathrm{kip/ft})}$$

$$E = 17.662\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.4223\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.30701\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (5.4223\ \mathrm{kipft/ft})) + (4 \times (-0.30701\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.0923\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.30701\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.662\ \mathrm{ft})}{(6\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.0923\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (17.662\ \mathrm{ft})}{(6\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.0923\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 7.2123\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.30701 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(17.662 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.662 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.662 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0923 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.964 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.022 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0035032 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.086 \text{ kipft}) + ((0.022 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013694 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013694 \text{ kipft/ft})}{(0.0035032 \text{ kip/ft})}$$

$$E = 3.9091 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013694 \text{ kipft/ft}) \times (6 \text{ ft})) + \left(3 \times (0.0035032 \text{ kip/ft}) \times (6 \text{ ft})^2\right)}{(6 \times (0.013694 \text{ kipft/ft})) + (4 \times (0.0035032 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2529 \text{ ft}$$

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

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

$$V_{max} = ((0.0035032 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2529 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2529 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.025455 \text{ kip}$$

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

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

$$M_{max} = ((0.0035032 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.9091 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.9091 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2529 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.068838 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(15.477 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.082 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.082 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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 $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(15.477\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0057854$$	Status: **PASS** Ratio: **0.010**

	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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 15.477\text{ kip} \rightarrow 15477\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(15477\text{ lbf})}{6 \times (2304\text{ in}^2)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.55\text{ kip}$$ 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 $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.55\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.55\text{ kip}$$	

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.55 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.44 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.2123 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.2123 \text{ kip})}{(111.44 \text{ kip})}$$

$$Ratio = 0.064721$$

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

Considering z-direction:

$V_{max} = 0.025455 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.025455 \text{ kip})}{(111.44 \text{ kip})}$$

$$Ratio = 0.00022842$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 20.964 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(20.964 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.083989$$	Status: **PASS** Ratio: **0.080**
	Considering z-direction: $M_{max} = 0.068838 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.068838 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.00027579$$	Status: **PASS** Ratio: **0.000**