

Project Name: MTSOLAR_H44ACJDCJ2G1-Tick Tock Energy
Date: Tue Feb 18 2025
Location: 1828 Olive Ave, Mattoon, IL 61938, USA
Number of Modules: 55
Unique ID: 4P-22.5-6TOP-XD-45-L-5Hx11W-E06J
Number of Poles: 4
Date Sold:
Dealer: _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	83.13 ft
Winter Tilt Angle	5
Front Edge Clearance	8 ft

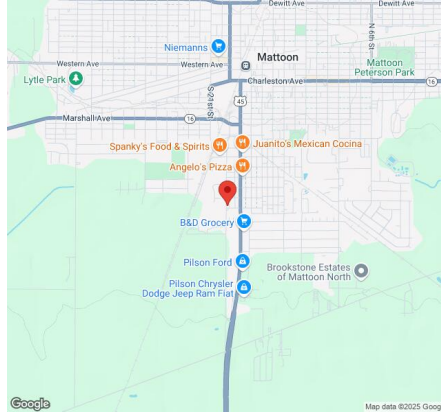
MT Solar Bill of Materials (4P-22.5-6TOP-XD-45-L-5Hx11W-E06J)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	12
MTS-CLAMP-ANGLE-4PK	Angle Clamp	11

Rail Bill of Materials

Part	Qty
Rails (223in)	22
Rail Attachment	88
Module Mid Clamp	88
Module End Clamp	44
Ground Lug	11

Site Details:



Site Address: 1828 Olive Ave, Mattoon, IL 61938, USA

Array Specification

Duty Classification:	XD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	5
Number of Columns:	11
Total Number of Modules:	55
Winter Tilt Angle:	5
Front Edge Clearance:	8
Total Array Height at Tilt:	9.64 ft
Total Frame Length:	82.50 ft
Frame Weight:	4906 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	83.13 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.82 ft
Number of Poles:	4
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.00 ft Pile 2: 5.25 ft Pile 3: 5.25 ft Pile 4: 5.00 ft
Foundation Volume:	12.148 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1828 Olive Ave, Mattoon, IL 61938, USA
Wind Speed:	100 mph

Snow Load:

20 psf

Design Disclaimer

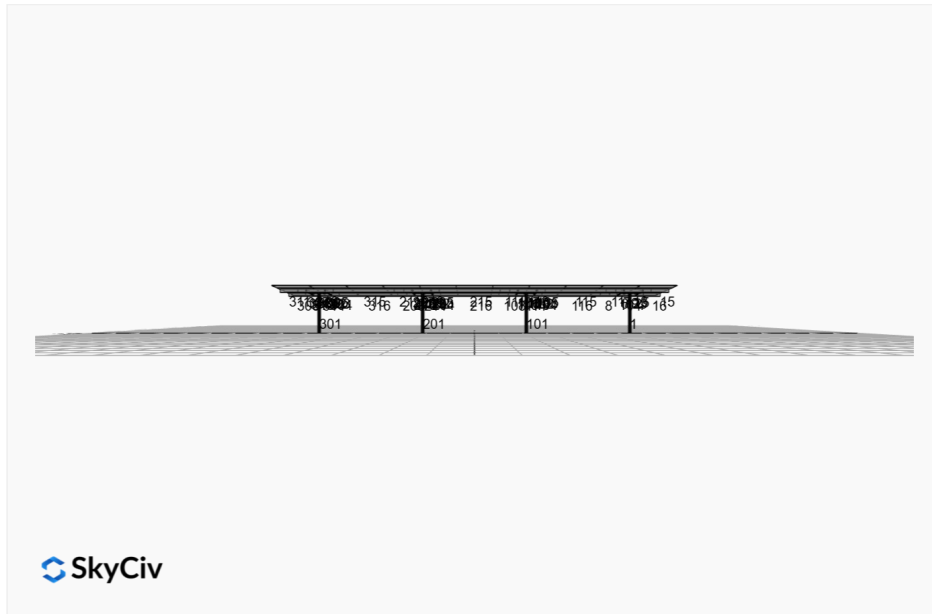
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

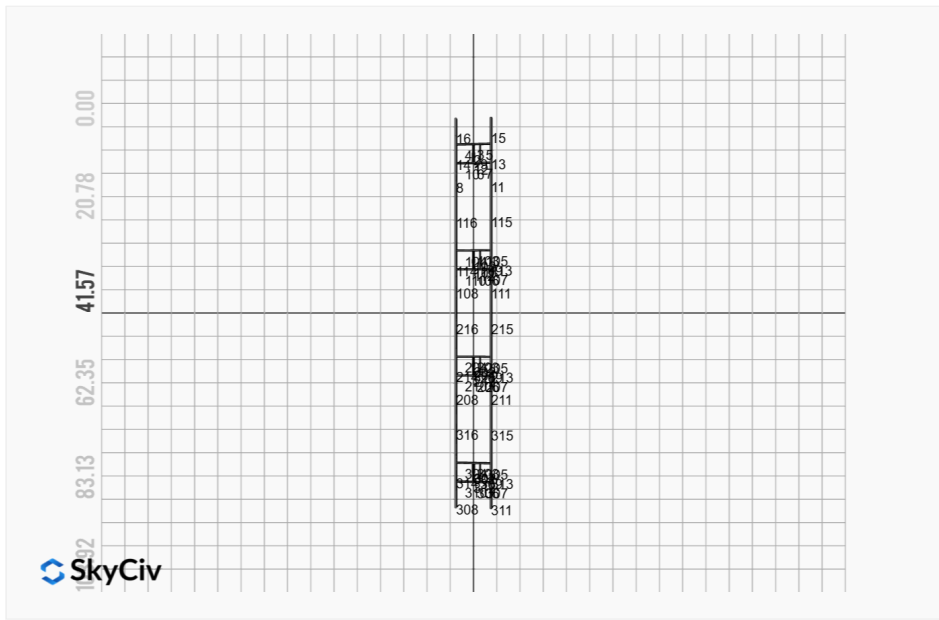
AutoDesigner Input

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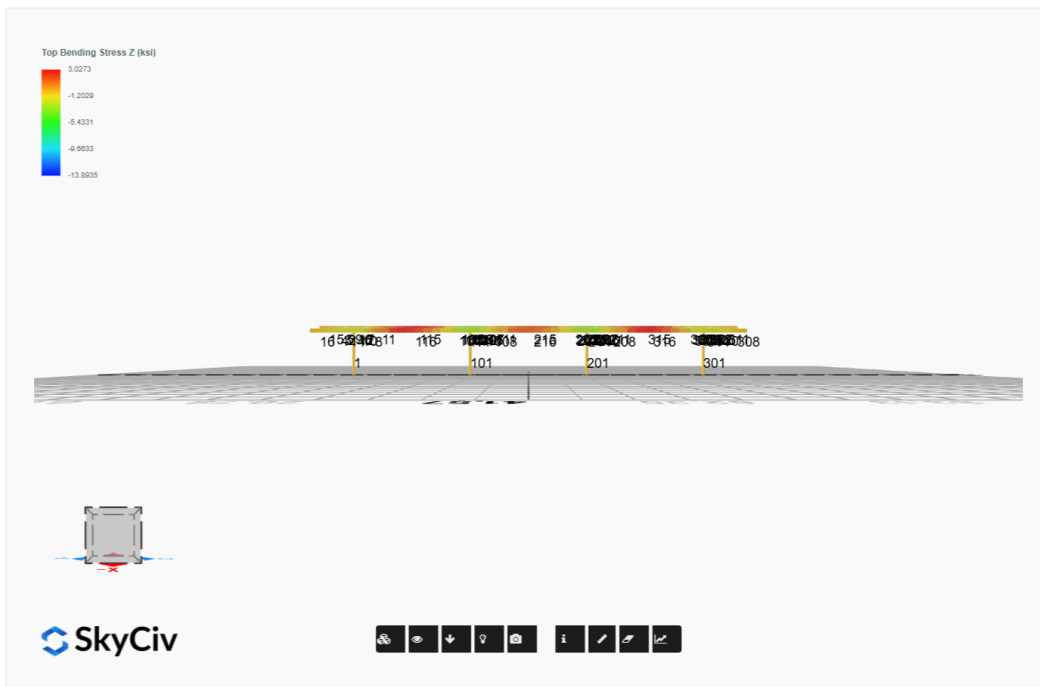
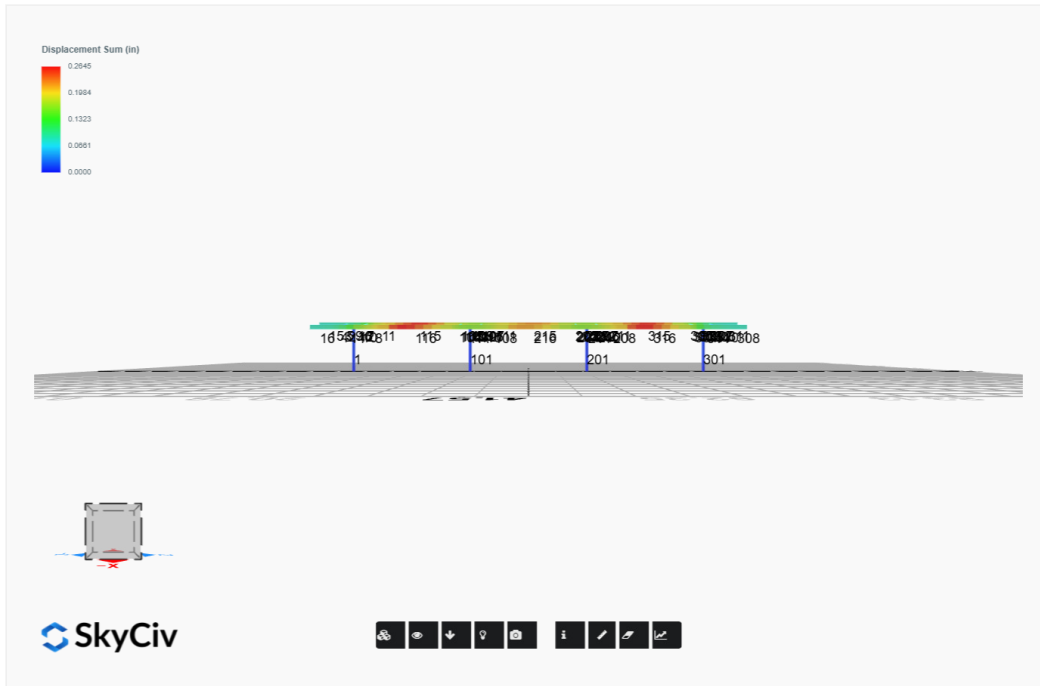
Design Notes:

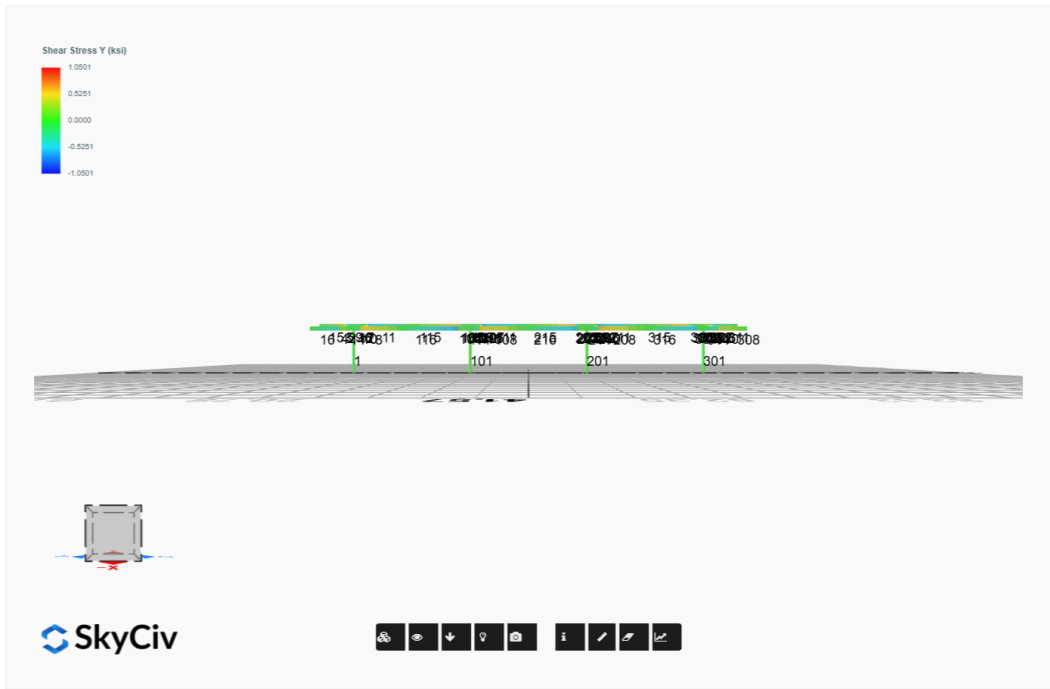
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigned are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel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

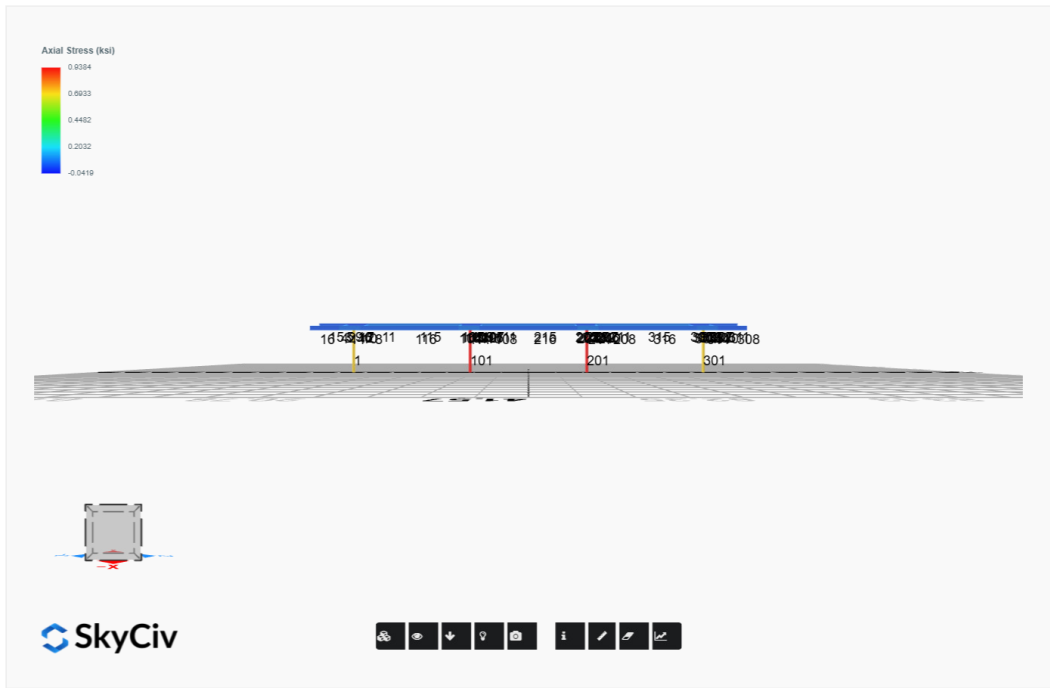




FEM Results (Envelope Worst Case for each member)







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	0.0052	2.3966	0.0727	0.1929	-0.0067	-0.0081
ULS: 2. D + L	0.0052	2.3966	0.0727	0.1929	-0.0067	-0.0081
ULS: 3. D + (S or Lr or R)	0.0160	6.5100	0.2260	0.6000	-0.0208	-0.0914
ULS: 3. D + (S or Lr or R)	0.0052	2.3966	0.0727	0.1929	-0.0067	-0.0081
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0133	5.4817	0.1877	0.4982	-0.0173	-0.0706
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0052	2.3966	0.0727	0.1929	-0.0067	-0.0081
ULS: 5b. D + 0.7E	0.0052	2.3966	0.0727	0.1929	-0.0067	-0.0081
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0133	5.4817	0.1877	0.4982	-0.0173	-0.0706
ULS: 8. 0.6D + 0.7E	0.0031	1.4380	0.0436	0.1157	-0.0040	-0.0049
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2888	5.7002	0.1977	0.5230	-0.0368	3.3259
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2888	5.7002	0.1977	0.5230	-0.0368	3.3259
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0778	1.5069	0.0425	0.1129	-0.0024	2.7856
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2045	0.2959	-0.0127	-0.0313	0.0196	-9.3377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2072	7.9594	0.2814	0.7458	-0.0399	2.4299
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2072	7.9594	0.2814	0.7458	-0.0399	2.4299
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0678	4.8144	0.1650	0.4383	-0.0140	2.0247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1628	3.9061	0.1236	0.3301	0.0024	-7.0678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2153	4.8743	0.1664	0.4404	-0.0293	2.4924
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2153	4.8743	0.1664	0.4404	-0.0293	2.4924
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0597	1.7293	0.0500	0.1329	-0.0034	2.0872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1547	0.8211	0.0087	0.0247	0.0130	-7.0053
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2909	4.7415	0.1686	0.4458	-0.0342	3.3291
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2909	4.7415	0.1686	0.4458	-0.0342	3.3291
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0758	0.5482	0.0134	0.0358	0.0003	2.7889
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2025	-0.6628	-0.0418	-0.1085	0.0223	-9.3344

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	12.2101
Shear X	-0.4899
Shear Z	0.4385
Moment X	1.1662
Moment Y (Twist)	0.0657
Moment Z	16.1271

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	7.9594
Shear X	-0.2909
Shear Z	0.2814
Moment X	0.7458
Moment Y (Twist)	0.0399
Moment Z	9.3377

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	-0.0052	2.9314	-0.0132	-0.0355	0.0028	0.0739
ULS: 2. D + L	-0.0052	2.9314	-0.0132	-0.0355	0.0028	0.0739
ULS: 3. D + (S or Lr or R)	-0.0160	8.1689	-0.0411	-0.1106	0.0087	0.1658
ULS: 3. D + (S or Lr or R)	-0.0052	2.9314	-0.0132	-0.0355	0.0028	0.0739
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0133	6.8595	-0.0341	-0.0918	0.0073	0.1428

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0052	2.9314	-0.0132	-0.0355	0.0028	0.0739
ULS: 5b. D + 0.7E	-0.0052	2.9314	-0.0132	-0.0355	0.0028	0.0739
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0133	6.8595	-0.0341	-0.0918	0.0073	0.1428
ULS: 8. 0.6D + 0.7E	-0.0031	1.7588	-0.0079	-0.0213	0.0017	0.0443
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3682	7.1378	-0.0348	-0.0939	0.0053	4.1155
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3682	7.1378	-0.0348	-0.0939	0.0053	4.1155
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0995	1.7946	-0.0057	-0.0156	-0.0011	3.2686
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2126	0.2638	-0.0027	-0.0066	0.0073	-10.8029
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2856	10.0143	-0.0503	-0.1356	0.0091	3.1741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2856	10.0143	-0.0503	-0.1356	0.0091	3.1741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0652	6.0069	-0.0285	-0.0769	0.0044	2.5389
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1500	4.8589	-0.0263	-0.0701	0.0106	-8.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2775	6.0862	-0.0294	-0.0793	0.0047	3.1051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2775	6.0862	-0.0294	-0.0793	0.0047	3.1051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0733	2.0788	-0.0076	-0.0206	-0.0001	2.4700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1582	0.9307	-0.0054	-0.0138	0.0062	-8.0837
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3662	5.9653	-0.0295	-0.0797	0.0042	4.0860
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3662	5.9653	-0.0295	-0.0797	0.0042	4.0860
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1015	0.6220	-0.0004	-0.0014	-0.0022	3.2391
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2147	-0.9087	0.0025	0.0076	0.0061	-10.8325

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.4033
Shear X	-0.6163
Shear Z	-0.0788
Moment X	-0.2131
Moment Y (Twist)	0.0175
Moment Z	18.6988

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	10.0143
Shear X	-0.3682
Shear Z	-0.0503
Moment X	-0.1356
Moment Y (Twist)	0.0106
Moment Z	10.8325

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	-0.0052	2.9314	0.0132	0.0355	-0.0028	0.0739
ULS: 2. D + L	-0.0052	2.9314	0.0132	0.0355	-0.0028	0.0739
ULS: 3. D + (S or Lr or R)	-0.0160	8.1689	0.0411	0.1106	-0.0087	0.1658
ULS: 3. D + (S or Lr or R)	-0.0052	2.9314	0.0132	0.0355	-0.0028	0.0739
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0133	6.8595	0.0341	0.0918	-0.0072	0.1428
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0052	2.9314	0.0132	0.0355	-0.0028	0.0739
ULS: 5b. D + 0.7E	-0.0052	2.9314	0.0132	0.0355	-0.0028	0.0739
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0133	6.8595	0.0341	0.0918	-0.0072	0.1428
ULS: 8. 0.6D + 0.7E	-0.0031	1.7588	0.0079	0.0213	-0.0017	0.0443
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3682	7.1378	0.0348	0.0939	-0.0053	4.1155
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3682	7.1378	0.0348	0.0939	-0.0053	4.1155
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0995	1.7946	0.0057	0.0156	0.0011	3.2686
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2126	0.2638	0.0027	0.0066	-0.0073	-10.8029

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	-0.2856	10.0143	0.0503	0.1356	-0.0091	3.1740
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2856	10.0143	0.0503	0.1356	-0.0091	3.1740
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0652	6.0069	0.0285	0.0769	-0.0043	2.5389
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1500	4.8589	0.0263	0.0701	-0.0106	-8.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2775	6.0862	0.0294	0.0793	-0.0047	3.1051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2775	6.0862	0.0294	0.0793	-0.0047	3.1051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0733	2.0788	0.0076	0.0206	0.0001	2.4700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1582	0.9307	0.0054	0.0138	-0.0062	-8.0837
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3662	5.9653	0.0295	0.0797	-0.0042	4.0860
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3662	5.9653	0.0295	0.0797	-0.0042	4.0860
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1015	0.6220	0.0004	0.0014	0.0022	3.2391
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2147	-0.9087	-0.0025	-0.0076	-0.0061	-10.8325

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.4033
Shear X	-0.6163
Shear Z	0.0788
Moment X	0.2131
Moment Y (Twist)	0.0173
Moment Z	18.6988

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	10.0143
Shear X	-0.3682
Shear Z	0.0503
Moment X	0.1356
Moment Y (Twist)	0.0106
Moment Z	10.8325

Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0052	2.3966	-0.0727	-0.1929	0.0067	-0.0081
ULS: 2. D + L	0.0052	2.3966	-0.0727	-0.1929	0.0067	-0.0081
ULS: 3. D + (S or Lr or R)	0.0160	6.5100	-0.2260	-0.6000	0.0209	-0.0914
ULS: 3. D + (S or Lr or R)	0.0052	2.3966	-0.0727	-0.1929	0.0067	-0.0081
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0133	5.4817	-0.1877	-0.4982	0.0173	-0.0706
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0052	2.3966	-0.0727	-0.1929	0.0067	-0.0081
ULS: 5b. D + 0.7E	0.0052	2.3966	-0.0727	-0.1929	0.0067	-0.0081
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0133	5.4817	-0.1877	-0.4982	0.0173	-0.0706
ULS: 8. 0.6D + 0.7E	0.0031	1.4380	-0.0436	-0.1157	0.0040	-0.0049
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2888	5.7002	-0.1977	-0.5230	0.0368	3.3259
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2888	5.7002	-0.1977	-0.5230	0.0368	3.3259
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0778	1.5069	-0.0425	-0.1129	0.0024	2.7856
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2045	0.2959	0.0127	0.0313	-0.0196	-9.3377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2072	7.9594	-0.2814	-0.7458	0.0399	2.4299
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2072	7.9594	-0.2814	-0.7458	0.0399	2.4299
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0678	4.8144	-0.1650	-0.4383	0.0141	2.0247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1628	3.9061	-0.1236	-0.3301	-0.0024	-7.0678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2153	4.8743	-0.1664	-0.4404	0.0293	2.4924
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2153	4.8743	-0.1664	-0.4404	0.0293	2.4924
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0597	1.7293	-0.0500	-0.1329	0.0034	2.0872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1547	0.8211	-0.0087	-0.0247	-0.0130	-7.0053

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2909	4.7415	-0.1686	-0.4458	0.0342	3.3291
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2909	4.7415	-0.1686	-0.4458	0.0342	3.3291
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0758	0.5482	-0.0134	-0.0358	-0.0003	2.7889
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2025	-0.6628	0.0418	0.1085	-0.0223	-9.3344

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.2101
Shear X	-0.4899
Shear Z	-0.4385
Moment X	-1.1663
Moment Y (Twist)	0.0658
Moment Z	16.1273

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	7.9594
Shear X	-0.2909
Shear Z	-0.2814
Moment X	-0.7458
Moment Y (Twist)	0.0399
Moment Z	9.3377

Project Details

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

User Name: sales@mtsolar.us
 Project Name: MTSOLAR_H44ACJDCJ2G1-Tick Tock Energy
 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_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.51	6.46	56.26	44.91
214	159.30	97.43	31.34	6.46	56.26	44.91
215	159.30	48.27	14.91	6.46	56.26	44.91
216	159.30	48.27	14.83	6.46	56.26	44.91
301	251.16	122.70	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	55.15	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	55.15	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	34.30	6.46	56.26	44.91
314	159.30	97.43	33.38	6.46	56.26	44.91
315	159.30	48.27	14.82	6.46	56.26	44.91
316	159.30	48.27	14.80	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	0.100	0.381	0.064	0.006	0.006	0.388	#16	0.495	Not Required	Pass
2	0.001	0.334	0.018	0.073	0.003	0.350	#21	0.036	Not Required	Pass
3	0.002	0.507	0.011	0.050	0.004	0.519	#21	0.046	Not Required	Pass
4	0.001	0.495	0.030	0.050	0.008	0.525	#21	0.122	Not Required	Pass
5	0.001	0.314	0.014	0.050	0.003	0.318	#21	0.076	Not Required	Pass
6	0.001	0.603	0.027	0.061	0.006	0.630	#21	0.046	Not Required	Pass
7	0.002	0.373	0.043	0.060	0.012	0.389	#21	0.076	Not Required	Pass
8	0.001	0.087	0.049	0.039	0.004	0.093	#21	0.102	Not Required	Pass
9	0.003	0.074	0.025	0.002	0.002	0.100	#21	0.206	Not Required	Pass
10	0.002	0.584	0.030	0.059	0.005	0.587	#21	0.082	Not Required	Pass
11	0.002	0.086	0.052	0.040	0.004	0.089	#21	0.102	Not Required	Pass
12	0.001	0.434	0.021	0.087	0.004	0.450	#21	0.054	Not Required	Pass
13	0.003	0.167	0.103	0.050	0.005	0.217	#21	0.306	Not Required	Pass
14	0.002	0.164	0.099	0.049	0.005	0.203	#21	0.204	Not Required	Pass
15	0.000	0.049	0.023	0.022	0.002	0.072	#21	Not Required	Not Required	Pass
16	0.000	0.048	0.023	0.021	0.002	0.071	#21	Not Required	Not Required	Pass
101	0.126	0.442	0.011	0.008	0.001	0.450	#16	0.495	Not Required	Pass
102	0.001	0.500	0.026	0.102	0.004	0.522	#21	0.036	Not Required	Pass
103	0.002	0.711	0.011	0.071	0.001	0.723	#21	0.046	Not Required	Pass
104	0.002	0.697	0.034	0.070	0.007	0.716	#21	0.082	Not Required	Pass
105	0.002	0.440	0.038	0.071	0.010	0.452	#21	0.076	Not Required	Pass
106	0.002	0.696	0.008	0.070	0.001	0.701	#21	0.046	Not Required	Pass
107	0.002	0.431	0.034	0.069	0.009	0.440	#21	0.076	Not Required	Pass
108	0.001	0.069	0.043	0.042	0.004	0.113	#21	0.102	Not Required	Pass
109	0.004	0.079	0.011	0.001	0.000	0.092	#21	0.206	Not Required	Pass
110	0.002	0.670	0.026	0.069	0.009	0.704	#21	0.082	Not Required	Pass

110	0.002	0.079	0.030	0.006	0.006	0.704	#21	0.002	Not Required	Pass
111	0.002	0.068	0.043	0.043	0.004	0.113	#21	0.102	Not Required	Pass
112	0.001	0.482	0.025	0.100	0.004	0.504	#21	0.054	Not Required	Pass
113	0.003	0.273	0.097	0.057	0.004	0.345	#21	0.306	Not Required	Pass
114	0.003	0.277	0.098	0.056	0.005	0.345	#21	0.306	Not Required	Pass
115	0.006	0.509	0.049	0.047	0.004	0.561	#21	0.644	Not Required	Pass
116	0.003	0.496	0.051	0.046	0.004	0.549	#21	0.644	Not Required	Pass
201	0.126	0.442	0.011	0.008	0.001	0.450	#16	0.495	Not Required	Pass
202	0.001	0.482	0.025	0.100	0.004	0.504	#21	0.054	Not Required	Pass
203	0.002	0.696	0.008	0.070	0.001	0.701	#21	0.046	Not Required	Pass
204	0.002	0.679	0.036	0.068	0.008	0.704	#21	0.082	Not Required	Pass
205	0.002	0.431	0.034	0.069	0.009	0.440	#21	0.076	Not Required	Pass
206	0.002	0.711	0.011	0.071	0.001	0.723	#21	0.046	Not Required	Pass
207	0.002	0.440	0.038	0.071	0.010	0.452	#21	0.076	Not Required	Pass
208	0.001	0.059	0.048	0.046	0.004	0.108	#21	0.102	Not Required	Pass
209	0.004	0.079	0.011	0.001	0.000	0.092	#21	0.206	Not Required	Pass
210	0.002	0.697	0.034	0.070	0.007	0.716	#21	0.082	Not Required	Pass
211	0.002	0.057	0.048	0.047	0.004	0.106	#21	0.102	Not Required	Pass
212	0.001	0.500	0.026	0.102	0.004	0.522	#21	0.036	Not Required	Pass
213	0.003	0.273	0.097	0.057	0.004	0.345	#21	0.306	Not Required	Pass
214	0.003	0.277	0.098	0.056	0.005	0.345	#21	0.306	Not Required	Pass
215	0.005	0.374	0.050	0.043	0.004	0.427	#21	0.644	Not Required	Pass
216	0.003	0.356	0.050	0.042	0.004	0.407	#21	0.644	Not Required	Pass
301	0.100	0.381	0.064	0.006	0.006	0.388	#16	0.495	Not Required	Pass
302	0.001	0.434	0.021	0.087	0.004	0.450	#21	0.054	Not Required	Pass
303	0.001	0.603	0.027	0.061	0.006	0.630	#21	0.046	Not Required	Pass
304	0.002	0.584	0.030	0.059	0.005	0.587	#21	0.082	Not Required	Pass
305	0.002	0.373	0.043	0.060	0.012	0.389	#21	0.076	Not Required	Pass
306	0.002	0.507	0.011	0.050	0.004	0.519	#21	0.046	Not Required	Pass
307	0.001	0.314	0.014	0.050	0.003	0.318	#21	0.076	Not Required	Pass
308	0.000	0.048	0.023	0.021	0.002	0.071	#21	Not Required	Not Required	Pass
309	0.003	0.074	0.025	0.002	0.002	0.100	#21	0.206	Not Required	Pass
310	0.001	0.495	0.030	0.050	0.008	0.525	#21	0.122	Not Required	Pass
311	0.000	0.049	0.023	0.022	0.002	0.072	#21	Not Required	Not Required	Pass
312	0.001	0.334	0.018	0.073	0.003	0.350	#21	0.036	Not Required	Pass
313	0.003	0.167	0.103	0.050	0.005	0.217	#21	0.204	Not Required	Pass
314	0.002	0.164	0.099	0.049	0.005	0.203	#21	0.306	Not Required	Pass
315	0.006	0.532	0.052	0.040	0.004	0.583	#21	0.644	Not Required	Pass
316	0.003	0.521	0.051	0.039	0.004	0.573	#21	0.644	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/compression)
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
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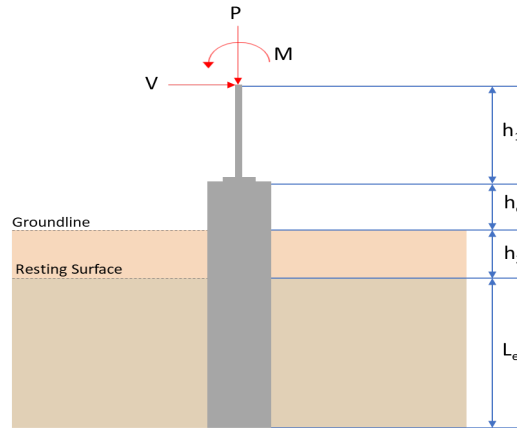
SkyCiv Foundation Design

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

$L = 5$ 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)	7.959	12.210
V_x (kip)	-0.291	-0.490
V_z (kip)	0.281	0.438
M_x (kipft)	0.746	1.166
M_z (kipft)	9.338	16.127

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,

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

$$H_o = \frac{(-0.291 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.338 \text{ kipft}) + ((-0.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4869 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7297 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.281 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.044745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.746 \text{ kipft}) + ((0.281 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11879 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.5362 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.7297 \text{ ft}), (2.5362 \text{ ft})]$$

$$L_{e,req} = 4.73 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.73 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.946$$

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

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

$$q = \frac{(7.959 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49744 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49744 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24872$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.046338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4869 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4869 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.4869 \text{ kipft/ft})) + (4 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3725 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4869 \text{ kipft/ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.4869 \text{ kipft/ft})) + (2 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.20706 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4869 \text{ kipft/ft})) + ((-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.65813 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3725 \text{ ft})}{2}$$

$$p_a = 0.25294 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20706 \text{ kip/ft}^2)}{(0.25294 \text{ kip/ft}^2)}$$

$$Ratio = 0.81863$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.65813 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.8775$$

Status: **PASS**
Ratio: **0.820**

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

Considering z-direction:

$H_o = 0.044745 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11879 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11879 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.044745 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.11879 \text{ kipft/ft})) + (4 \times (0.044745 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5653 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11879 \text{ kipft/ft})) + (3 \times (0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.11879 \text{ kipft/ft})) + (2 \times (0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.049044 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.11879 \text{ kipft/ft})) + ((0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.11071 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5653 \text{ ft})}{2}$$

$$p_a = 0.2674 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.049044 \text{ kip/ft}^2)}{(0.2674 \text{ kip/ft}^2)}$$

$$Ratio = 0.18341$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.11071 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.14762$$

Status: **PASS**
Ratio: **0.180**

Status: **PASS**
Ratio: **0.150**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.49 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.078025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.127 \text{ kipft}) + ((-0.49 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.568 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.568 \text{ kipft/ft})}{(-0.078025 \text{ kip/ft})}$$

$$E = 32.912 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.568 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.568 \text{ kipft/ft})) + (4 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(6 \times (2.568 \text{ kipft/ft})) + (4 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft}))}{}$$

$$a = 3.3717 \text{ ft}$$

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

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

$$V_{max} = ((-0.078025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3717 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3717 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.8504 \text{ kip}$$

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

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

$$M_{max} = ((-0.078025 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(32.912 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.4824 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.438 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.069745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.166 \text{ kipft}) + ((0.438 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.18567 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.18567 \text{ kipft/ft})}{(0.069745 \text{ kip/ft})}$$

$$E = 2.6621 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18567 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.069745 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.18567 \text{ kipft/ft})) + (4 \times (0.069745 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.565 \text{ ft}$$

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

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

$$V_{max} = ((0.069745 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.565 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.565 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44853 \text{ kip}$$

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

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

$$M_{max} = ((0.069745 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(2.6621 \text{ ft})}{(5 \text{ ft})} + \frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.99681 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.21 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.19 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.19 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(12.21 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0045642$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.21 \text{ kip} \rightarrow 12210 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12210 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.11 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.11 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.11 \text{ kip}$$

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.11 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.15 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.8504 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.8504 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.03464$$

Considering z-direction:

$V_{max} = 0.44853 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.44853 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.0040352$$

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

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:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 9.4824 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.4824 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.03799$$

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

Considering z-direction:

$M_{max} = 0.99681 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.99681 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0039936$$

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

REFERENCES	CALCULATIONS	RESULTS
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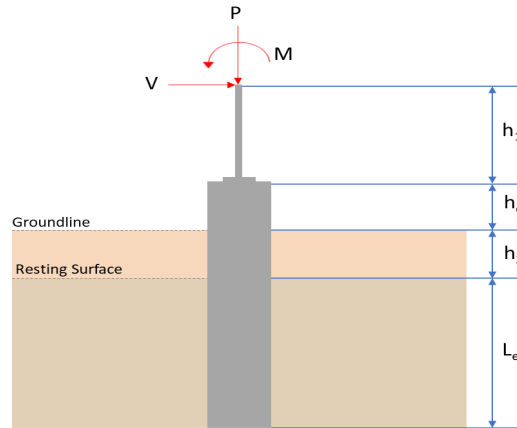
SkyCiv Foundation Design

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

$L = 5$ 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)	7.959	12.210
V_x (kip)	-0.291	-0.490
V_z (kip)	-0.281	-0.438
M_x (kipft)	-0.746	-1.166
M_z (kipft)	9.338	16.127

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,

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

$$H_o = \frac{(-0.291 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.338 \text{ kipft}) + ((-0.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4869 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7297 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.281 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.746 \text{ kipft}) + ((-0.281 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11879 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.7023 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.7297 \text{ ft}), (1.7023 \text{ ft})]$$

$$L_{e,req} = 4.73 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.73 \text{ ft})}{(5 \text{ ft})}$$

$$Ratio = 0.946$$

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

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

$$q = \frac{(7.959 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49744 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49744 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24872$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.046338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4869 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4869 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.4869 \text{ kipft/ft})) + (4 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3725 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4869 \text{ kipft/ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.4869 \text{ kipft/ft})) + (2 \times (-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.20706 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4869 \text{ kipft/ft})) + ((-0.046338 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.65813 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3725 \text{ ft})}{2}$$

$$p_a = 0.25294 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20706 \text{ kip/ft}^2)}{(0.25294 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.81863$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.65813 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8775$$

Status: **PASS**
Ratio: **0.820**

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

Considering z-direction:

$H_o = -0.044745 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11879 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11879 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.044745 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.11879 \text{ kipft/ft})) + (4 \times (-0.044745 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5653 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11879 \text{ kipft/ft})) + (3 \times (-0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.11879 \text{ kipft/ft})) + (2 \times (-0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = -0.012656 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.11879 \text{ kipft/ft})) + ((-0.044745 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.0033248 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5653 \text{ ft})}{2}$$

$$p_a = 0.2674 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.012656 \text{ kip/ft}^2)}{(0.2674 \text{ kip/ft}^2)}$$

$$Ratio = -0.047329$$

Status: **PASS**
Ratio: **-0.050**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0033248 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.0044331$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.49 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.078025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.127 \text{ kipft}) + ((-0.49 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.568 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.568 \text{ kipft/ft})}{(-0.078025 \text{ kip/ft})}$$

$$E = 32.912 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.568 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.568 \text{ kipft/ft})) + (4 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(6 \times (2.568 \text{ kipft/ft})) + (4 \times (-0.078025 \text{ kip/ft}) \times (5 \text{ ft}))}{}$$

$$a = 3.3717 \text{ ft}$$

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

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

$$V_{max} = ((-0.078025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3717 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3717 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.8504 \text{ kip}$$

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

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

$$M_{max} = ((-0.078025 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(32.912 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (32.912 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3717 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.4824 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.438 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.069745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.166 \text{ kipft}) + ((-0.438 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.18567 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.18567 \text{ kipft/ft})}{(-0.069745 \text{ kip/ft})}$$

$$E = 2.6621 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18567 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.069745 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.18567 \text{ kipft/ft})) + (4 \times (-0.069745 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.565 \text{ ft}$$

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

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

$$V_{max} = ((-0.069745 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.565 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.565 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44853 \text{ kip}$$

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

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

$$M_{max} = ((-0.069745 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(2.6621 \text{ ft})}{(5 \text{ ft})} + \frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.565 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.99681 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.21 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.19 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.19 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(12.21 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0045642$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.21 \text{ kip} \rightarrow 12210 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12210 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.11 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.11 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.11 \text{ kip}$$

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.11 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.15 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.8504 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.8504 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.03464$$

Considering z-direction:

$V_{max} = 0.44853 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.44853 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.0040352$$

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

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:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 9.4824 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.4824 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.03799$$

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

Considering z-direction:

$M_{max} = 0.99681 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.99681 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0039936$$

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

REFERENCES	CALCULATIONS	RESULTS
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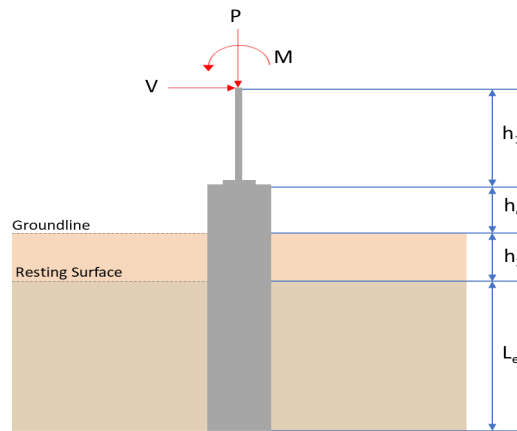
SkyCiv Foundation Design

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

$L = 5.25$ 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)	10.014	15.403
V_x (kip)	-0.368	-0.616
V_z (kip)	-0.050	-0.079
M_x (kipft)	-0.136	-0.213
M_z (kipft)	10.832	18.699

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,

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

$$H_o = \frac{(-0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.058599 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.832 \text{ kipft}) + ((-0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7248 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.9408 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.05 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.136 \text{ kipft}) + ((-0.05 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.021656 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.0692 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9408 \text{ ft}), (1.0692 \text{ ft})]$$

$$L_{e,req} = 4.941 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.941 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.94114$$

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

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

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

$$q = \frac{(10.014 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.62587 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62587 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.31294$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.058599 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7248 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.7248 \text{ kipft/ft})) + (4 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5465 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7248 \text{ kipft/ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (1.7248 \text{ kipft/ft})) + (2 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.21317 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.7248 \text{ kipft/ft})) + ((-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.68398 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5465 \text{ ft})}{2}$$

$$p_a = 0.26599 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21317 \text{ kip/ft}^2)}{(0.26599 \text{ kip/ft}^2)}$$

$$Ratio = 0.80145$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.68398 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.86855$$

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

Status: **PASS**
Ratio: **0.870**

Considering z-direction:

$H_o = -0.0079618 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.021656 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021656 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.021656 \text{ kipft/ft})) + (4 \times (-0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7462 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.021656 \text{ kipft/ft})) + (3 \times (-0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.021656 \text{ kipft/ft})) + (2 \times (-0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = -0.0021958 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.021656 \text{ kipft/ft})) + ((-0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.0003293 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7462 \text{ ft})}{2}$$

$$p_a = 0.28096 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0021958 \text{ kip/ft}^2)}{(0.28096 \text{ kip/ft}^2)}$$

$$Ratio = -0.0078153$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0003293 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.00041816$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.616 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.098089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.699 \text{ kipft}) + ((-0.616 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9775 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.9775 \text{ kipft/ft})}{(-0.098089 \text{ kip/ft})}$$

$$E = 30.356 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9775 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.9775 \text{ kipft/ft})) + (4 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(-0.098089 \text{ kip/ft}) \times (48 \text{ in}) + (4 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft}))}{(6 \times (2.9775 \text{ kipft/ft})) + (4 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5452 \text{ ft}$$

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

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

$$V_{max} = ((-0.098089 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2824 \text{ kip}$$

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

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

$$M_{max} = ((-0.098089 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(30.356 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.052 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.079 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01258 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.213 \text{ kipft}) + ((-0.079 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.033917 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.033917 \text{ kipft/ft})}{(-0.01258 \text{ kip/ft})}$$

$$E = 2.6962 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.033917 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.01258 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.033917 \text{ kipft/ft})) + (4 \times (-0.01258 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7471 \text{ ft}$$

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

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

$$V_{max} = ((-0.01258 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7471 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7471 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.079239 \text{ kip}$$

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

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

$$M_{max} = ((-0.01258 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.6962 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.18443 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.403 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.084 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.084 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio - Capacity</i></p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(15.403 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0057577$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.403 \text{ kip} \rightarrow 15403 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15403 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.43 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.2824 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2824 \text{ kip})}{(111.43 \text{ kip})}$$

$$Ratio = 0.038431$$

Considering z-direction:

$V_{max} = 0.079239 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.079239 \text{ kip})}{(111.43 \text{ kip})}$$

$$Ratio = 0.0007111$$

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

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:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 11.052 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(11.052 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.044279$$

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

Considering z-direction:

$M_{max} = 0.18443 \text{ kipft}$ - Maximum moment in the z-direction,

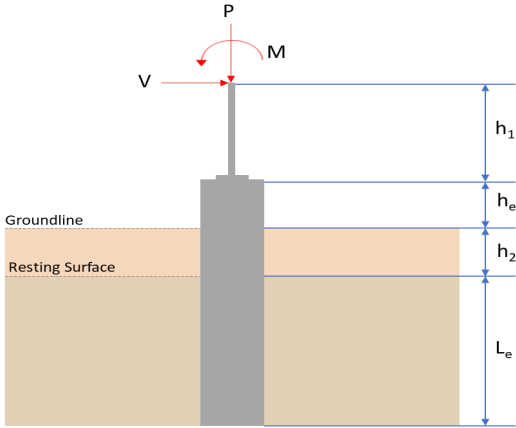
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.18443 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00073891$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.014</td> <td>15.403</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.368</td> <td>-0.616</td> </tr> <tr> <td>V_z (kip)</td> <td>0.050</td> <td>0.079</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.136</td> <td>0.213</td> </tr> <tr> <td>M_z (kipft)</td> <td>10.832</td> <td>18.699</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength.</p>	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	Load Component	ASD	LRFD	P (kip)	10.014	15.403	V_x (kip)	-0.368	-0.616	V_z (kip)	0.050	0.079	M_x (kipft)	0.136	0.213	M_z (kipft)	10.832	18.699	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	10.832	18.699																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.058599 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.832 \text{ kipft}) + ((-0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7248 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.9408 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.05 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.136 \text{ kipft}) + ((0.05 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.021656 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.3332 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.9408 \text{ ft}), (1.3332 \text{ ft})]$$

$$L_{e,req} = 4.941 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.941 \text{ ft})}{(5.25 \text{ ft})}$$

$$Ratio = 0.94114$$

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

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

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

$$q = \frac{(10.014 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.62587 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62587 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.31294$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.058599 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7248 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.7248 \text{ kipft/ft})) + (4 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5465 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7248 \text{ kipft/ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (1.7248 \text{ kipft/ft})) + (2 \times (-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.21317 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.7248 \text{ kipft/ft})) + ((-0.058599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.68398 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5465 \text{ ft})}{2}$$

$$p_a = 0.26599 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21317 \text{ kip/ft}^2)}{(0.26599 \text{ kip/ft}^2)}$$

$$Ratio = 0.80145$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.68398 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.86855$$

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

Status: **PASS**
Ratio: **0.870**

Considering z-direction:

$H_o = 0.0079618 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.021656 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021656 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.021656 \text{ kipft/ft})) + (4 \times (0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7462 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.021656 \text{ kipft/ft})) + (3 \times (0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.021656 \text{ kipft/ft})) + (2 \times (0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.0082335 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.021656 \text{ kipft/ft})) + ((0.0079618 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.018528 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7462 \text{ ft})}{2}$$

$$p_a = 0.28096 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0082335 \text{ kip/ft}^2)}{(0.28096 \text{ kip/ft}^2)}$$

$$Ratio = 0.029304$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.018528 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.023527$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.616 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.098089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.699 \text{ kipft}) + ((-0.616 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9775 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.9775 \text{ kipft/ft})}{(-0.098089 \text{ kip/ft})}$$

$$E = 30.356 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9775 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.9775 \text{ kipft/ft})) + (4 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(-0.098089 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.9775 \text{ kipft/ft})) + (4 \times (-0.098089 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5452 \text{ ft}$$

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

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

$$V_{max} = ((-0.098089 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2824 \text{ kip}$$

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

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

$$M_{max} = ((-0.098089 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(30.356 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (30.356 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.052 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.079 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.01258 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.213 \text{ kipft}) + ((0.079 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.033917 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.033917 \text{ kipft/ft})}{(0.01258 \text{ kip/ft})}$$

$$E = 2.6962 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.033917 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.01258 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.033917 \text{ kipft/ft})) + (4 \times (0.01258 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7471 \text{ ft}$$

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

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

$$V_{max} = ((0.01258 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7471 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7471 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.079239 \text{ kip}$$

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

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

$$M_{max} = ((0.01258 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.6962 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6962 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7471 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.18443 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.403 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.084 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.084 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(15.403 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0057577$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.403 \text{ kip} \rightarrow 15403 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15403 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.43 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.2824 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2824 \text{ kip})}{(111.43 \text{ kip})}$$

$$Ratio = 0.038431$$

Considering z-direction:

$V_{max} = 0.079239 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.079239 \text{ kip})}{(111.43 \text{ kip})}$$

$$Ratio = 0.0007111$$

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

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:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 11.052 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.052 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.044279$$

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

Considering z-direction:

$M_{max} = 0.18443 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.18443 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00073891$$

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