

Project Name: MTSOLAR_Honeywell Aguadilla Clay 4x12 **Date:** Fri Mar 28 2025

4P - V1JB

Number of Modules: 48

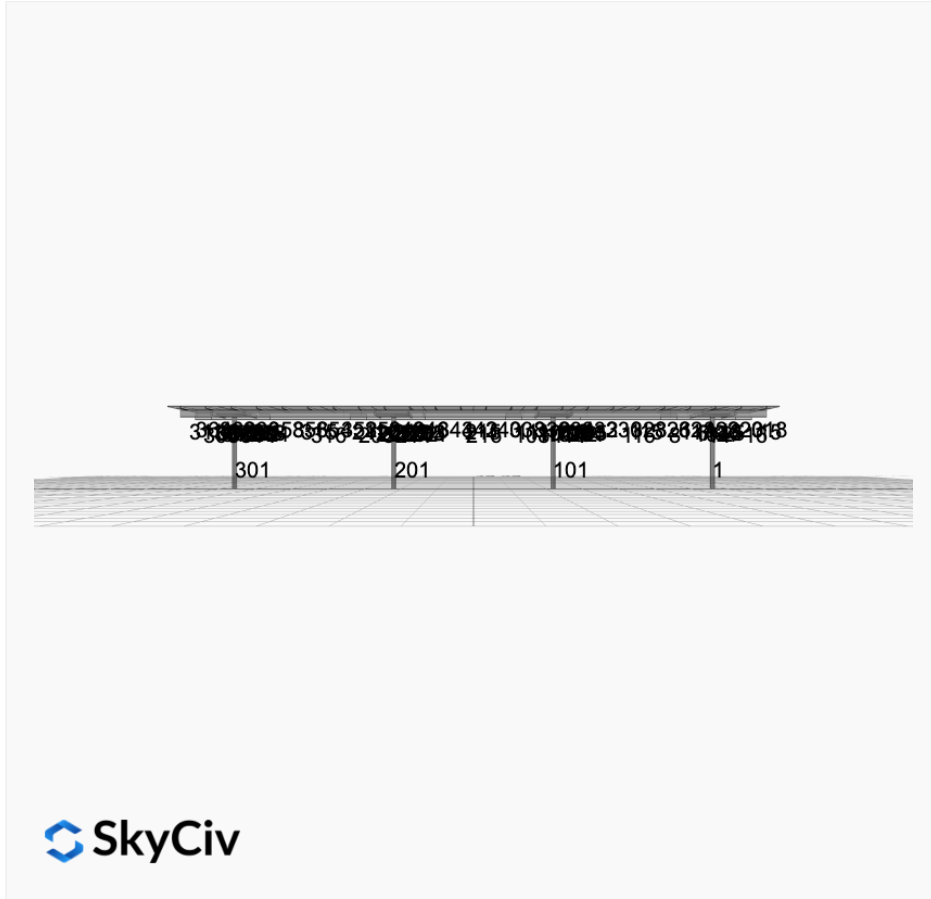
Location: FHMW+FW, Homestead Base, FL, USA

Number of Poles: 4

Unique ID: 4P-19.75-6TOP-XD-24-L-4Hx12W-AFF7

Date Sold:

Dealer: _____



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	71.90 ft
Winter Tilt Angle	1
Front Edge Clearance	9 ft

MT Solar Bill of Materials (4P-19.75-6TOP-XD-24-L-4Hx12W-AFF7)

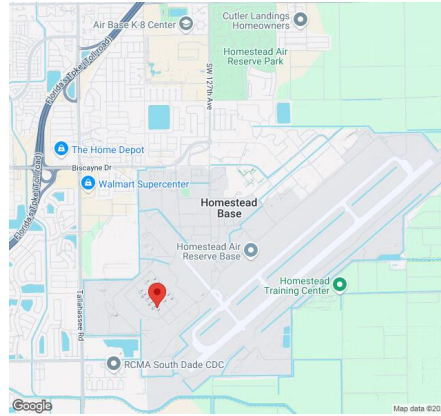
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-24	24IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-HOOK-4PK	Hook Clamp	12

Rail Bill of Materials

Part	Qty
Rails (180in)	24
Rail Attachment	48

Part	Qty
Module Mid Clamp	72
Module End Clamp	48
Ground Lug	12

Site Details:



Site Address: FHMW+FW, Homestead Base, FL, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	70.90in
Number of Rows:	4
Number of Columns:	12
Total Number of Modules:	48
Winter Tilt Angle:	1
Front Edge Clearance:	9
Total Array Height at Tilt:	9.26 ft
Total Frame Length:	70.75 ft
Module Info/Notes:	
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	71.90 ft
Rail Length:	180.40 in
Rail Spacing:	3.00 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.13 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft Pile 2: 6.75 ft Pile 3: 6.75 ft Pile 4: 6.50 ft
Foundation Volume:	15.704 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	clay
Site Location:	FHMW+FW, Homestead Base, FL, USA
Wind Speed:	175 mph

Snow Load:

0 psf

Design Disclaimer

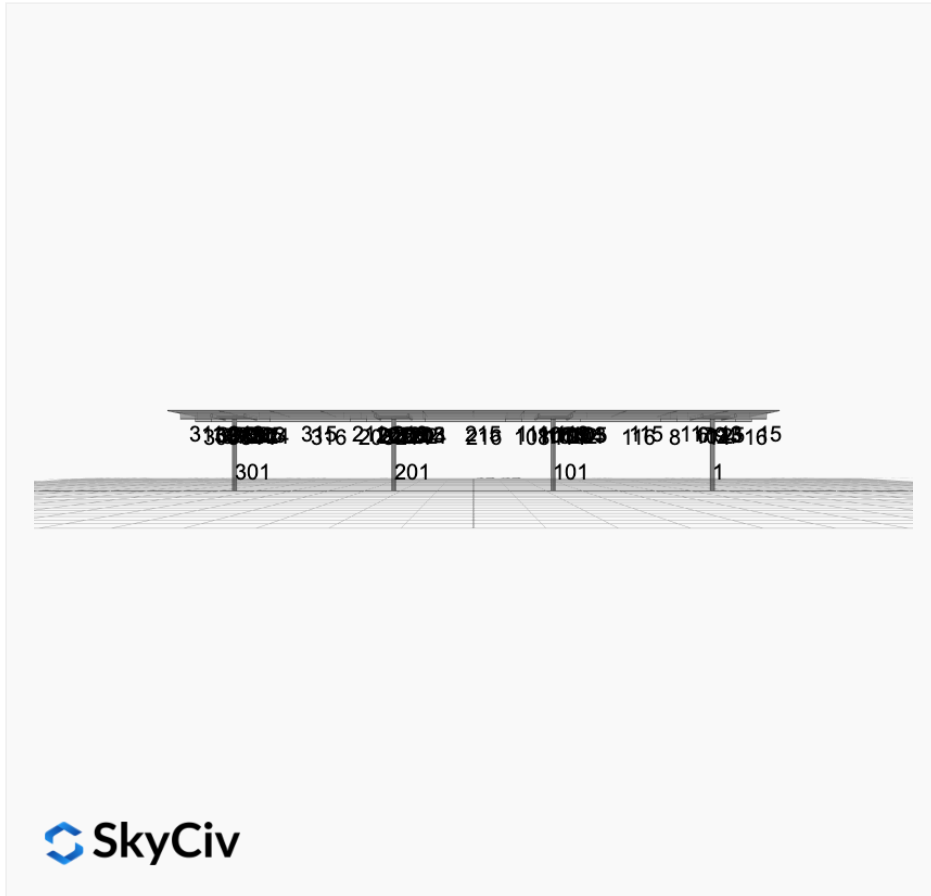
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

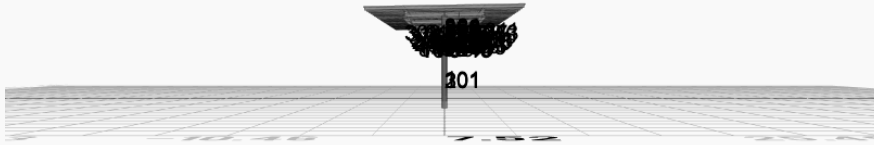
AutoDesigner Input

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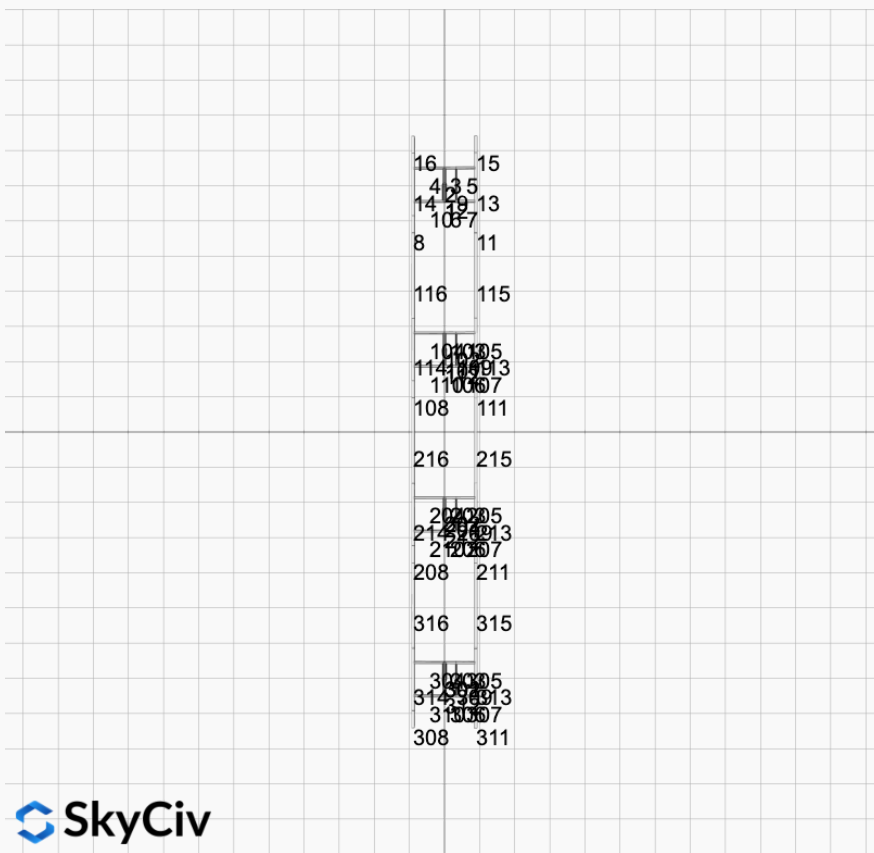
Design Notes:

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





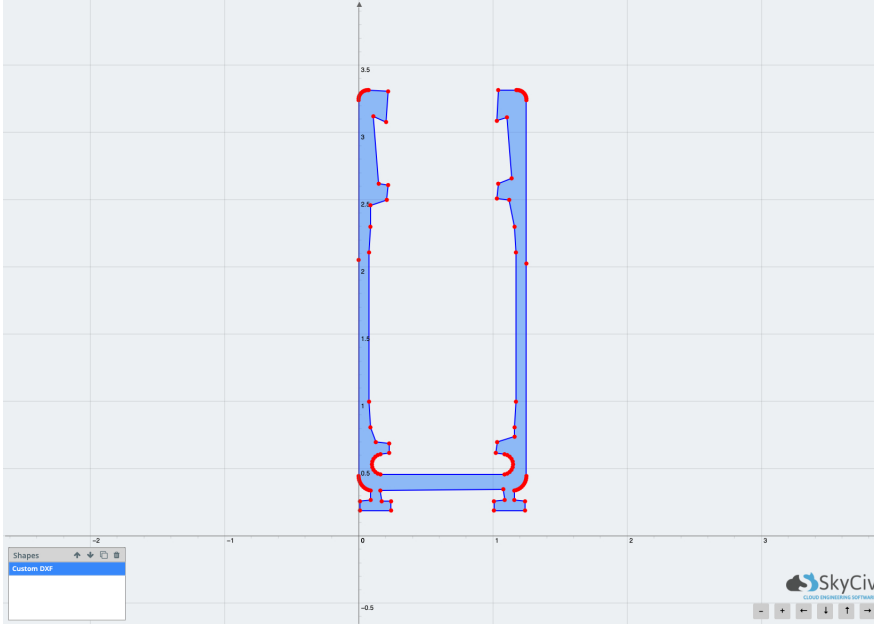
 SkyCiv



 SkyCiv

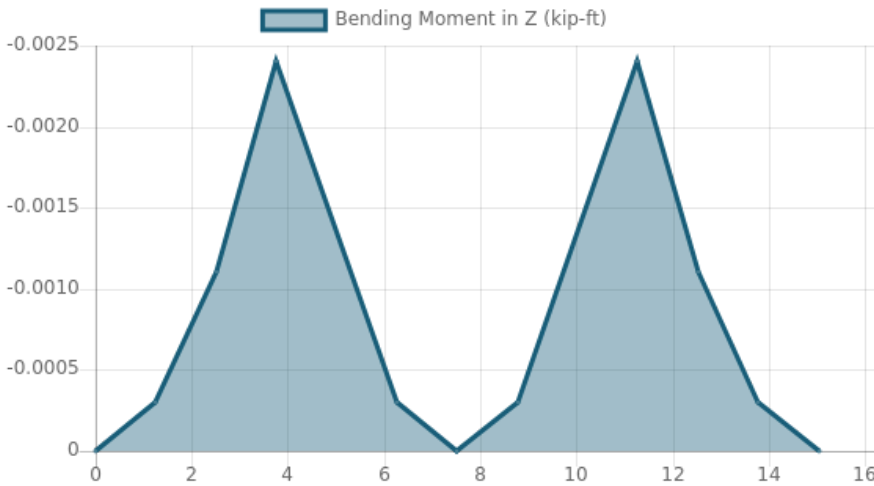
Rail Design Check

Rail Length: 15.033333333333333 ft
Additional Restraints Required: None
Tributary Width: 2.9958333333333336 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Wind uplift Case A (X): 0.0000 kip/ft
Wind uplift Case A (Y): -0.0182 kip/ft
Wind uplift Case A: -0.1383 kip/ft
Wind uplift Case B: -0.1383 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1642 kip/ft

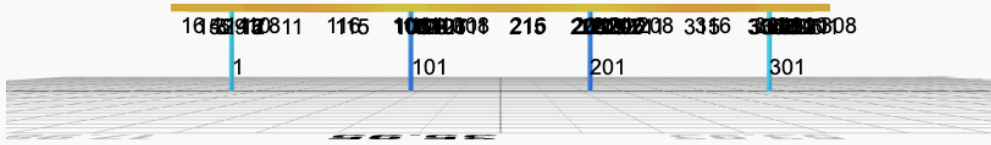


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	25.49540705	0.739	PASS
Material Yield	34.5	25.49540705	0.739	PASS
Material Strength	37	25.49540705	0.689	PASS

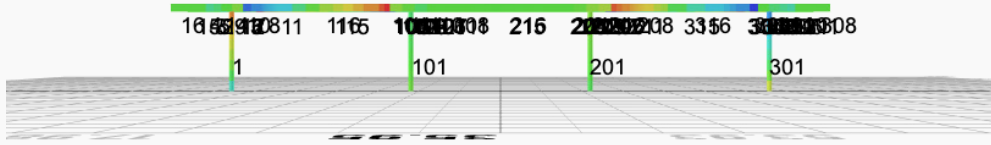
Member 1, ULS: 1. 1.4D



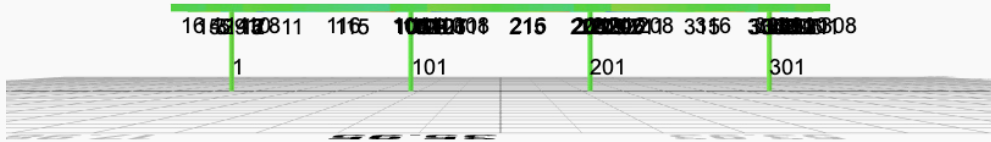
Top Bending Stress Z (ksi)



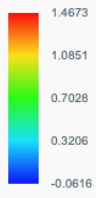
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 2, D + L	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 3, D + (S or Lr or R)	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 3, D + (S or Lr or R)	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 5b, D + 0.7E	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 6b, D + 0.75L + 0.75(0.7)E + 0.75S	0.0012	1.8598	0.0649	0.1799	-0.0028	0.0235
ULS: 8, 0.6D + 0.7E	0.0007	1.1159	0.0389	0.1080	-0.0017	0.0141
ULS: 5a, D + 0.6W_Wind downforce Case A only	-0.0577	7.1541	0.3049	0.8485	-0.0052	-9.1735
ULS: 5a, D + 0.6W_Wind downforce Case B only	-0.0577	7.1541	0.3049	0.8485	-0.0052	-9.1735
ULS: 5a, D + 0.6W_Wind uplift Case A only	-0.1041	5.4161	0.2344	0.6487	-0.0320	12.4047
ULS: 5a, D + 0.6W_Wind uplift Case B only	0.1177	-2.1538	-0.1245	-0.3413	0.0290	-14.0665
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0430	5.8305	0.2449	0.6814	-0.0046	-6.8743
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0430	5.8305	0.2449	0.6814	-0.0046	-6.8743
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0778	4.5270	0.1920	0.5315	-0.0247	9.3094
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0885	-1.1504	-0.0772	-0.2110	0.0210	-10.5440
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0430	5.8305	0.2449	0.6814	-0.0046	-6.8743
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0430	5.8305	0.2449	0.6814	-0.0046	-6.8743
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0778	4.5270	0.1920	0.5315	-0.0247	9.3094
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0885	-1.1504	-0.0772	-0.2110	0.0210	-10.5440
ULS: 7, 0.6D + 0.6W_Wind downforce Case A only	-0.0581	6.4102	0.2789	0.7766	-0.0041	-9.1829
ULS: 7, 0.6D + 0.6W_Wind downforce Case B only	-0.0581	6.4102	0.2789	0.7766	-0.0041	-9.1829
ULS: 7, 0.6D + 0.6W_Wind uplift Case A only	-0.1046	4.6722	0.2085	0.5768	-0.0309	12.3953
ULS: 7, 0.6D + 0.6W_Wind uplift Case B only	0.1172	-2.8977	-0.1505	-0.4133	0.0301	-14.0759

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	11.0554
Shear X	-0.1973
Shear Z	0.4785
Moment X	1.3339
Moment Y (Twist)	0.0529
Moment Z	23.8559

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	7.1541
Shear X	-0.1177
Shear Z	0.3049
Moment X	0.8485
Moment Y (Twist)	0.0320
Moment Z	14.0759

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	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 2, D + L	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 3, D + (S or Lr or R)	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 3, D + (S or Lr or R)	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 5b. D + 0.7E	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0011	2.4023	-0.0104	-0.0291	0.0006	0.0414
ULS: 8. 0.6D + 0.7E	-0.0007	1.4414	-0.0062	-0.0175	0.0004	0.0248
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1627	9.7336	-0.0513	-0.1423	0.0165	-10.8305
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1627	9.7336	-0.0513	-0.1423	0.0165	-10.8305
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0437	7.3151	-0.0341	-0.0972	-0.0146	14.7756
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0492	-3.1443	0.0174	0.0500	0.0167	-16.3990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1223	7.9008	-0.0410	-0.1140	0.0126	-8.1125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1223	7.9008	-0.0410	-0.1140	0.0126	-8.1125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0331	6.0869	-0.0281	-0.0801	-0.0108	11.0920
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0366	-1.7577	0.0104	0.0302	0.0127	-12.2889
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1223	7.9008	-0.0410	-0.1140	0.0126	-8.1125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1223	7.9008	-0.0410	-0.1140	0.0126	-8.1125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0331	6.0869	-0.0281	-0.0801	-0.0108	11.0920
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0366	-1.7577	0.0104	0.0302	0.0127	-12.2889
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1622	8.7727	-0.0471	-0.1306	0.0163	-10.8471
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1622	8.7727	-0.0471	-0.1306	0.0163	-10.8471
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0432	6.3542	-0.0299	-0.0855	-0.0148	14.7590
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0497	-4.1053	0.0215	0.0616	0.0164	-16.4156

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	15.1019
Shear X	-0.2720
Shear Z	-0.0808
Moment X	-0.2245
Moment Y (Twist)	0.0283
Moment Z	27.8842

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.7336
Shear X	-0.1627
Shear Z	-0.0513
Moment X	-0.1423
Moment Y (Twist)	0.0167
Moment Z	16.4156

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	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 2. D + L	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 3. D + (S or Lr or R)	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 3. D + (S or Lr or R)	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 5b. D + 0.7E	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0012	2.4027	0.0107	0.0301	-0.0007	0.0414
ULS: 8. 0.6D + 0.7E	-0.0007	1.4416	0.0064	0.0181	-0.0004	0.0249
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1627	9.7354	0.0525	0.1471	-0.0159	-10.8323
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1627	9.7354	0.0525	0.1471	-0.0159	-10.8323
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0437	7.3164	0.0350	0.1007	0.0137	14.7779
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0493	-3.1450	-0.0178	-0.0518	-0.0157	-16.4014

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	-0.1223	7.9022	0.0420	0.1179	-0.0121	-8.1139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1223	7.9022	0.0420	0.1179	-0.0121	-8.1139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0331	6.0880	0.0289	0.0831	0.0101	11.0938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0367	-1.7581	-0.0107	-0.0313	-0.0120	-12.2907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1223	7.9022	0.0420	0.1179	-0.0121	-8.1139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1223	7.9022	0.0420	0.1179	-0.0121	-8.1139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0331	6.0880	0.0289	0.0831	0.0101	11.0938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0367	-1.7581	-0.0107	-0.0313	-0.0120	-12.2907
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1622	8.7744	0.0482	0.1351	-0.0156	-10.8489
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1622	8.7744	0.0482	0.1351	-0.0156	-10.8489
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0433	6.3553	0.0307	0.0887	0.0139	14.7614
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0497	-4.1061	-0.0221	-0.0639	-0.0155	-16.4180

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	15.1047
Shear X	-0.2720
Shear Z	0.0827
Moment X	0.2322
Moment Y (Twist)	0.0267
Moment Z	27.8884

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.7354
Shear X	-0.1627
Shear Z	0.0525
Moment X	0.1471
Moment Y (Twist)	0.0159
Moment Z	16.4180

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	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 2. D + L	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 3. D + (S or Lr or R)	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 3. D + (S or Lr or R)	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 5b. D + 0.7E	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0012	1.8597	-0.0651	-0.1804	0.0028	0.0235
ULS: 8. 0.6D + 0.7E	0.0007	1.1158	-0.0391	-0.1082	0.0017	0.0141
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0577	7.1536	-0.3061	-0.8504	0.0056	-9.1783
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0577	7.1536	-0.3061	-0.8504	0.0056	-9.1783
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.1041	5.4157	-0.2353	-0.6502	0.0319	12.4103
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1176	-2.1536	0.1250	0.3421	-0.0288	-14.0731
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0430	5.8301	-0.2459	-0.6829	0.0049	-6.8779
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0430	5.8301	-0.2459	-0.6829	0.0049	-6.8779
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0778	4.5267	-0.1928	-0.5327	0.0247	9.3136
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0885	-1.1503	0.0775	0.2115	-0.0209	-10.5489
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0430	5.8301	-0.2459	-0.6829	0.0049	-6.8779
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0430	5.8301	-0.2459	-0.6829	0.0049	-6.8779
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0778	4.5267	-0.1928	-0.5327	0.0247	9.3136
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0885	-1.1503	0.0775	0.2115	-0.0209	-10.5489

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0582	6.4097	-0.2801	-0.7783	0.0044	-9.1877
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0582	6.4097	-0.2801	-0.7783	0.0044	-9.1877
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.1046	4.6718	-0.2093	-0.5780	0.0308	12.4009
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1171	-2.8975	0.1511	0.4142	-0.0299	-14.0825

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	11.0546
Shear X	-0.1973
Shear Z	-0.4804
Moment X	-1.3368
Moment Y (Twist)	0.0527
Moment Z	23.8669

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	7.1536
Shear X	-0.1176
Shear Z	-0.3061
Moment X	-0.8504
Moment Y (Twist)	0.0319
Moment Z	14.0825

Project Details

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

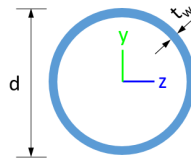


Design Input Information

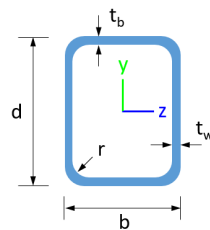
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

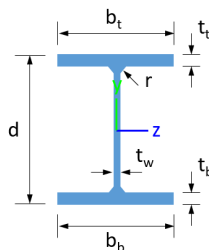
Section Dimensions



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



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------

315	20	10.2 0	10.2 0	10. 20	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.11,1.11,1.09,1.09,1.11,1.11,1.08,1.08,1.11,1.12,1.08,1.08,1.11,1.12,1.09,1.09,1.11,1.11	30 0	20 0	1
316	20	10.2 0	10.2 0	10. 20	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.13,1.83,1.10,1.10,1.13,1.83,1.10,1.10,1.10,1.21,1.10,1.10,1.10,1.21,1.10,1.10,1.14,2.03	30 0	20 0	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	116.55	42.30	42.30	75.35	75.35
2	251.01	229.64	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	39.18	6.46	56.26	44.91
14	159.30	97.43	38.87	6.46	56.26	44.91
15	159.30	113.66	46.90	6.46	56.26	44.91
16	159.30	113.66	46.90	6.46	56.26	44.91
101	251.16	116.55	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.95	6.46	56.26	44.91
114	159.30	97.43	30.80	6.46	56.26	44.91
115	159.30	32.87	21.57	6.46	56.26	44.91
116	159.30	32.87	20.76	6.46	56.26	44.91
201	251.16	116.55	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30

212	251.01	248.88	27.10	27.10	75.30	75.30
213	159.30	97.43	31.94	6.46	56.26	44.91
214	159.30	97.43	30.80	6.46	56.26	44.91
215	159.30	32.87	22.07	6.46	56.26	44.91
216	159.30	32.87	19.63	6.46	56.26	44.91
301	251.16	116.55	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	113.66	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	113.66	46.90	6.46	56.26	44.91
312	251.01	229.64	27.16	27.16	75.30	75.30
313	159.30	97.43	39.15	6.46	56.26	44.91
314	159.30	97.43	38.85	6.46	56.26	44.91
315	159.30	32.87	20.83	6.46	56.26	44.91
316	159.30	32.87	21.09	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	0.095	0.564	0.072	0.003	0.006	0.589	#32	0.512	Not Required	Pass
2	0.001	0.288	0.004	0.064	0.001	0.292	#13	0.174	Not Required	Pass
3	0.001	0.521	0.020	0.051	0.006	0.529	#15	0.046	Not Required	Pass
4	0.001	0.607	0.020	0.061	0.004	0.627	#13	0.122	Not Required	Pass
5	0.001	0.320	0.009	0.051	0.004	0.330	#15	0.076	Not Required	Pass
6	0.001	0.650	0.020	0.067	0.004	0.662	#15	0.046	Not Required	Pass
7	0.001	0.405	0.025	0.065	0.008	0.410	#15	0.076	Not Required	Pass
8	0.002	0.148	0.039	0.048	0.001	0.164	#13	0.102	Not Required	Pass
9	0.002	0.062	0.019	0.003	0.002	0.082	#13	0.206	Not Required	Pass
10	0.001	0.758	0.025	0.076	0.006	0.772	#13	0.082	Not Required	Pass
11	0.002	0.122	0.037	0.042	0.001	0.144	#15	0.102	Not Required	Pass
12	0.001	0.399	0.008	0.080	0.002	0.404	#13	0.054	Not Required	Pass
13	0.003	0.104	0.049	0.054	0.002	0.124	#15	0.306	Not Required	Pass
14	0.003	0.121	0.052	0.063	0.002	0.147	#13	0.204	Not Required	Pass
15	0.000	0.017	0.000	0.015	0.000	0.018	#15	Not Required	Not Required	Pass
16	0.000	0.021	0.000	0.017	0.000	0.021	#13	Not Required	Not Required	Pass
101	0.130	0.673	0.012	0.004	0.001	0.693	#32	0.512	Not Required	Pass
102	0.000	0.488	0.009	0.100	0.003	0.497	#13	0.054	Not Required	Pass
103	0.000	0.773	0.005	0.077	0.001	0.778	#15	0.046	Not Required	Pass
104	0.001	0.919	0.016	0.092	0.003	0.928	#13	0.122	Not Required	Pass
105	0.000	0.479	0.019	0.077	0.005	0.483	#15	0.076	Not Required	Pass
106	0.000	0.775	0.007	0.078	0.003	0.782	#15	0.046	Not Required	Pass
107	0.000	0.482	0.003	0.077	0.001	0.483	#15	0.076	Not Required	Pass
108	0.002	0.066	0.002	0.053	0.000	0.067	#13	0.102	Not Required	Pass
109	0.002	0.075	0.007	0.001	0.001	0.076	#13	0.206	Not Required	Pass
110	0.000	0.914	0.005	0.091	0.001	0.919	#13	0.122	Not Required	Pass

111	0.002	0.063	0.001	0.045	0.000	0.064	#15	0.102	Not Required	Pass
112	0.000	0.470	0.008	0.098	0.001	0.477	#13	0.054	Not Required	Pass
113	0.004	0.203	0.048	0.060	0.001	0.220	#15	0.306	Not Required	Pass
114	0.004	0.262	0.049	0.072	0.001	0.265	#13	0.306	Not Required	Pass
115	0.010	0.384	0.036	0.048	0.001	0.392	#15	0.780	Not Required	Pass
116	0.008	0.449	0.037	0.057	0.001	0.453	#13	0.780	Not Required	Pass
201	0.130	0.673	0.012	0.004	0.001	0.693	#32	0.512	Not Required	Pass
202	0.000	0.470	0.008	0.098	0.001	0.477	#13	0.054	Not Required	Pass
203	0.000	0.775	0.007	0.078	0.003	0.782	#15	0.046	Not Required	Pass
204	0.000	0.914	0.005	0.091	0.001	0.919	#13	0.122	Not Required	Pass
205	0.000	0.482	0.003	0.077	0.001	0.483	#15	0.076	Not Required	Pass
206	0.000	0.773	0.005	0.077	0.001	0.778	#15	0.046	Not Required	Pass
207	0.000	0.479	0.019	0.077	0.005	0.483	#15	0.076	Not Required	Pass
208	0.002	0.102	0.037	0.057	0.001	0.122	#13	0.102	Not Required	Pass
209	0.002	0.076	0.007	0.001	0.001	0.076	#13	0.206	Not Required	Pass
210	0.001	0.919	0.016	0.092	0.003	0.929	#13	0.122	Not Required	Pass
211	0.002	0.092	0.036	0.048	0.001	0.116	#15	0.102	Not Required	Pass
212	0.000	0.489	0.009	0.100	0.003	0.498	#13	0.054	Not Required	Pass
213	0.004	0.203	0.048	0.060	0.001	0.220	#15	0.306	Not Required	Pass
214	0.004	0.263	0.049	0.072	0.001	0.266	#13	0.306	Not Required	Pass
215	0.011	0.285	0.002	0.045	0.000	0.287	#15	0.780	Not Required	Pass
216	0.010	0.316	0.002	0.053	0.000	0.318	#13	0.780	Not Required	Pass
301	0.095	0.564	0.072	0.003	0.006	0.589	#32	0.512	Not Required	Pass
302	0.001	0.399	0.008	0.080	0.002	0.404	#13	0.054	Not Required	Pass
303	0.001	0.651	0.020	0.067	0.005	0.662	#15	0.046	Not Required	Pass
304	0.001	0.759	0.025	0.076	0.006	0.773	#13	0.082	Not Required	Pass
305	0.001	0.406	0.025	0.065	0.008	0.411	#15	0.076	Not Required	Pass
306	0.001	0.521	0.021	0.051	0.006	0.529	#15	0.046	Not Required	Pass
307	0.001	0.320	0.009	0.051	0.004	0.330	#15	0.076	Not Required	Pass
308	0.000	0.021	0.000	0.017	0.000	0.021	#13	Not Required	Not Required	Pass
309	0.002	0.062	0.019	0.003	0.002	0.082	#13	0.206	Not Required	Pass
310	0.001	0.606	0.020	0.061	0.004	0.627	#13	0.122	Not Required	Pass
311	0.000	0.017	0.000	0.015	0.000	0.018	#15	Not Required	Not Required	Pass
312	0.001	0.287	0.004	0.064	0.001	0.292	#13	0.174	Not Required	Pass
313	0.003	0.104	0.049	0.054	0.002	0.124	#15	0.204	Not Required	Pass
314	0.003	0.121	0.052	0.063	0.002	0.148	#13	0.306	Not Required	Pass
315	0.010	0.395	0.037	0.042	0.001	0.399	#15	0.780	Not Required	Pass
316	0.008	0.467	0.039	0.048	0.001	0.472	#13	0.780	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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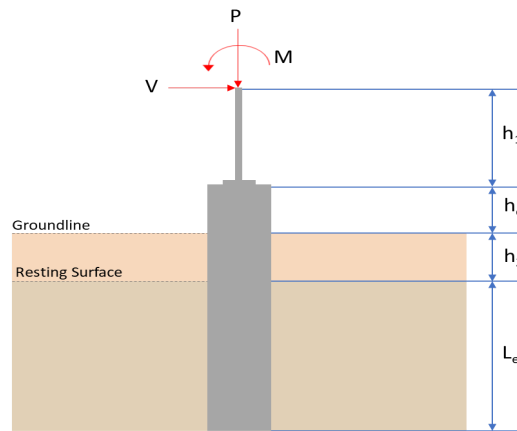
SkyCiv Foundation Design

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

$L = 6.5$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500,000	100,000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.154	11.055
V_x (kip)	-0.118	-0.197
V_z (kip)	0.305	0.478
M_x (kipft)	0.849	1.334
M_z (kipft)	14.076	23.856

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,

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

$$H_o = \frac{(-0.118 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.076 \text{ kipft}) + ((-0.118 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.2414 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.3677 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.305 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.048567 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.849 \text{ kipft}) + ((0.305 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13519 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.0993 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.3677 \text{ ft}), (3.0993 \text{ ft})]$$

$$L_{e,req} = 6.368 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.368 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.9797$$

Status: **PASS**
Ratio: **0.980**

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

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

$$q = \frac{(7.154 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44712 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44712 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.29808$$

Status: **PASS**
Ratio: **0.300**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.01879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.2414 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.2414 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (2.2414 \text{ kipft/ft})) + (4 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3523 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.2414 \text{ kipft/ft})) + (3 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (2.2414 \text{ kipft/ft})) + (2 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.20257 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.2414 \text{ kipft/ft})) + ((-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.61927 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.3523 \text{ ft})}{2}$$

$$p_a = 0.21762 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20257 \text{ kip/ft}^2)}{(0.21762 \text{ kip/ft}^2)}$$

$$Ratio = 0.93087$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.65 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.61927 \text{ kip/ft}^2)}{(0.65 \text{ kip/ft}^2)}$$

$$Ratio = 0.95272$$

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

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

Considering z-direction:

$H_o = 0.048567 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.13519 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13519 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.048567 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.13519 \text{ kipft/ft})) + (4 \times (0.048567 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6631 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.13519 \text{ kipft/ft})) + (3 \times (0.048567 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.13519 \text{ kipft/ft})) + (2 \times (0.048567 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.037895 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.13519 \text{ kipft/ft})) + ((0.048567 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.083228 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.6631 \text{ ft})}{2}$$

$$p_a = 0.23316 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.037895 \text{ kip/ft}^2)}{(0.23316 \text{ kip/ft}^2)}$$

$$Ratio = 0.16253$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.65 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

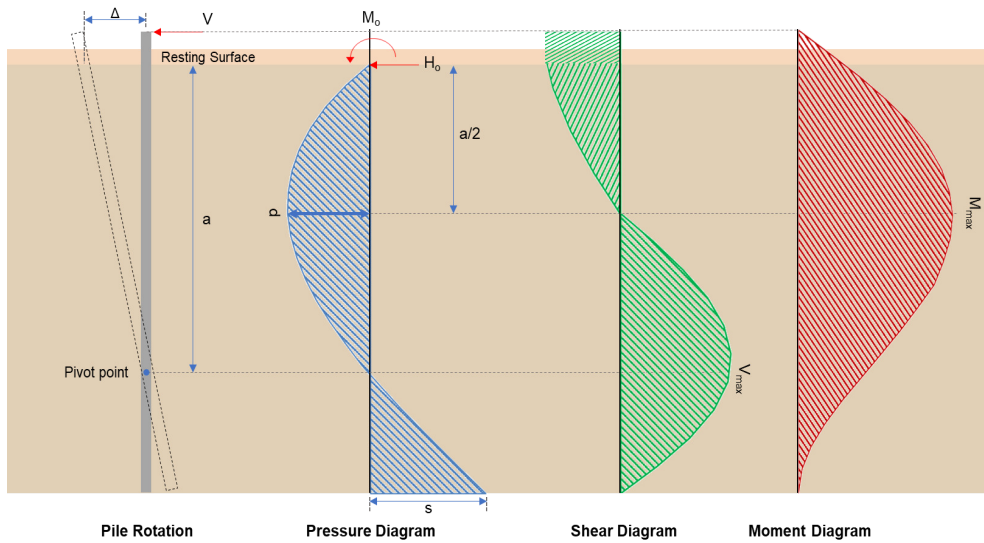
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.083228 \text{ kip/ft}^2)}{(0.65 \text{ kip/ft}^2)}$$

$$Ratio = 0.12804$$

Status: **PASS**
Ratio: **0.160**

Status: **PASS**
Ratio: **0.130**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.197 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.031369 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(23.856 \text{ kipft}) + ((-0.197 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.7987 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.7987 \text{ kipft/ft})}{(-0.031369 \text{ kip/ft})}$$

$$E = 121.1 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.7987 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.031369 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.7987 \text{ kipft/ft})) + (4 \times (-0.031369 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = \frac{(-0.031369 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (3.7987 \text{ kip/ft})) + (4 \times (-0.031369 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.352 \text{ ft}$$

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

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

$$V_{max} = ((-0.031369 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (121.1 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.352 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (121.1 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.352 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2351 \text{ kip}$$

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

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

$$M_{max} = ((-0.031369 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(121.1 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.352 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (121.1 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.352 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (121.1 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.352 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.689 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.478 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.076115 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.334 \text{ kipft}) + ((0.478 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21242 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21242 \text{ kipft/ft})}{(0.076115 \text{ kip/ft})}$$

$$E = 2.7908 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21242 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.076115 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.21242 \text{ kipft/ft})) + (4 \times (0.076115 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6628 \text{ ft}$$

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

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

$$V_{max} = ((0.076115 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7908 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6628 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.7908 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6628 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.43464 \text{ kip}$$

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

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

$$M_{max} = ((0.076115 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.7908 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6628 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.7908 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6628 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7908 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6628 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2364 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{11.055 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.229 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.229 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 s_{rebar} - Minimum spacing of reinforcement,</p> <p>25.7.2.2 Since longitudinal reinforcement is \leq No. 10Ø: Use #3(0.375 in)</p> <p>25.7.2.1 s_{ties} - Maximum spacing of ties,</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p style="text-align: center;">$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p style="text-align: center;">$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</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 ϕP_N - Allowable axial compressive strength</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.055 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0041324$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2 b_w = 48 in - Effective width, d - Effective depth</p> <p>22.5.5.1.3 λ_s - size effect modification factor</p> <p>22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</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 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 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.055 \text{ kip} \rightarrow 11055 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11055 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.96 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.96 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.96 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.96 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.05 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.2351 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2351 \text{ kip})}{(111.05 \text{ kip})}$$

$$Ratio = 0.038135$$

Considering z-direction:

$V_{max} = 0.43464 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.43464 \text{ kip})}{(111.05 \text{ kip})}$$

$$Ratio = 0.0039137$$

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:

$M_{max} = 13.689 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.689 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.054843$$

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

Considering z-direction:

$M_{max} = 1.2364 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.2364 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0049536$$

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

REFERENCES	CALCULATIONS	RESULTS
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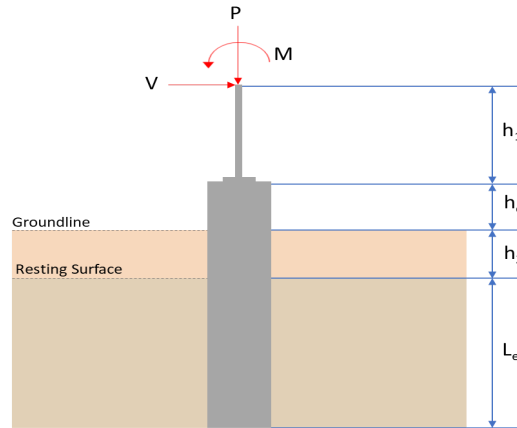
SkyCiv Foundation Design

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

$L = 6.5$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500,000	100,000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.154	11.055
V_x (kip)	-0.118	-0.197
V_z (kip)	-0.306	-0.480
M_x (kipft)	-0.850	-1.337
M_z (kipft)	14.082	23.867

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,

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

$$H_o = \frac{(-0.118 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.082 \text{ kipft}) + ((-0.118 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.2424 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.3687 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.306 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.048726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.85 \text{ kipft}) + ((-0.306 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13535 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9673 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.3687 \text{ ft}), (1.9673 \text{ ft})]$$

$$L_{e,req} = 6.369 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.369 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.97985$$

Status: **PASS**
Ratio: **0.980**

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

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

$$q = \frac{(7.154 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44712 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44712 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.29808$$

Status: **PASS**
Ratio: **0.300**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.01879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.2424 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.2424 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (2.2424 \text{ kipft/ft})) + (4 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3523 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.2424 \text{ kipft/ft})) + (3 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (2.2424 \text{ kipft/ft})) + (2 \times (-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.20266 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.2424 \text{ kipft/ft})) + ((-0.01879 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.61954 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.3523 \text{ ft})}{2}$$

$$p_a = 0.21762 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20266 \text{ kip/ft}^2)}{(0.21762 \text{ kip/ft}^2)}$$

$$Ratio = 0.93129$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.65 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.61954 \text{ kip/ft}^2)}{(0.65 \text{ kip/ft}^2)}$$

$$Ratio = 0.95314$$

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

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

Considering z-direction:

$H_o = -0.048726 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.13535 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13535 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.048726 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.13535 \text{ kipft/ft})) + (4 \times (-0.048726 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6634 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.13535 \text{ kipft/ft})) + (3 \times (-0.048726 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (0.13535 \text{ kipft/ft})) + (2 \times (-0.048726 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = -0.013044 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.13535 \text{ kipft/ft})) + ((-0.048726 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = -0.0065353 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.6634 \text{ ft})}{2}$$

$$p_a = 0.23317 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.013044 \text{ kip/ft}^2)}{(0.23317 \text{ kip/ft}^2)}$$

$$Ratio = -0.05594$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.65 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

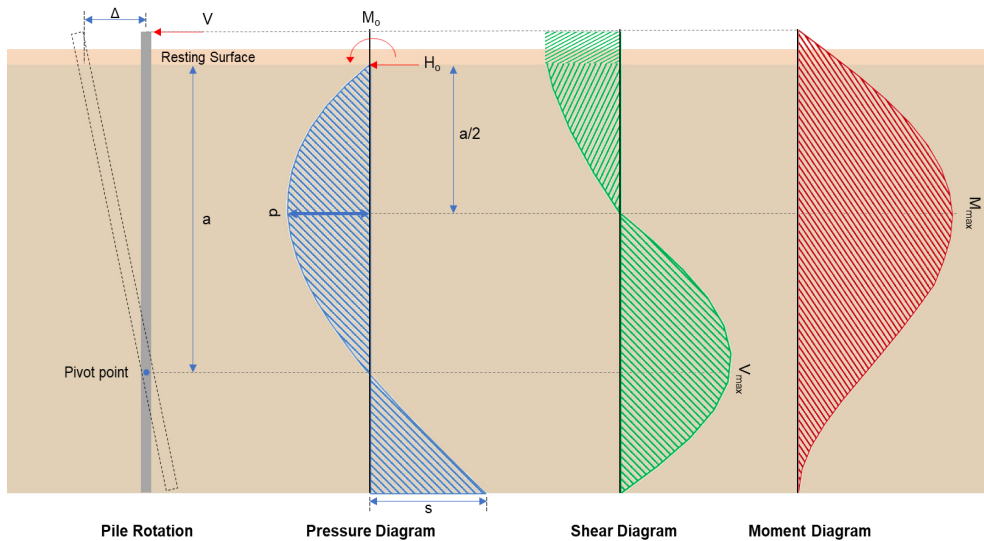
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0065353 \text{ kip/ft}^2)}{(0.65 \text{ kip/ft}^2)}$$

$$Ratio = -0.010054$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.197 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.031369 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(23.867 \text{ kipft}) + ((-0.197 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8005 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.8005 \text{ kipft/ft})}{(-0.031369 \text{ kip/ft})}$$

$$E = 121.15 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8005 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.031369 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times 3.8005) + (4 \times (-0.031369) \times 6.5)}$$

$$a = \frac{(-0.031369 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})}{(6 \times (3.8005 \text{ kipft/ft})) + (4 \times (-0.031369 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4,352 \text{ ft}$$

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

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

$$V_{max} = ((-0.031369 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (121,15 \text{ ft})}{(6,5 \text{ ft})} + 3 \right) \times \left(\frac{(4,352 \text{ ft})}{(6,5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (121,15 \text{ ft})}{(6,5 \text{ ft})} + 2 \right) \times \left(\frac{(4,352 \text{ ft})}{(6,5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4,237 \text{ kip}$$

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

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

$$M_{max} = ((-0.031369 \text{ kip/ft}) \times (48 \text{ in}) \times (6,5 \text{ ft})) \times \left[\left(\frac{(121,15 \text{ ft})}{(6,5 \text{ ft})} + \frac{(4,352 \text{ ft})}{2 \times (6,5 \text{ ft})} \right) - \left[\left(\frac{4 \times (121,15 \text{ ft})}{(6,5 \text{ ft})} + 3 \right) \times \left(\frac{(4,352 \text{ ft})}{2 \times (6,5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (121,15 \text{ ft})}{(6,5 \text{ ft})} + 2 \right) \times \left(\frac{(4,352 \text{ ft})}{2 \times (6,5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13,695 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1,57 b}$$

$$H_o = \frac{(-0,48 \text{ kip})}{1,57 \times (48 \text{ in})}$$

$$H_o = -0,076433 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{1,57 b}$$

$$M_o = \frac{(1,337 \text{ kipft}) + ((-0,48 \text{ kip}) \times (0 \text{ ft}))}{1,57 \times (48 \text{ in})}$$

$$M_o = 0,2129 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0,2129 \text{ kipft/ft})}{(-0,076433 \text{ kip/ft})}$$

$$E = 2,7854 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0,2129 \text{ kipft/ft}) \times (6,5 \text{ ft})) + (3 \times (-0,076433 \text{ kip/ft}) \times (6,5 \text{ ft})^2)}{(6 \times (0,2129 \text{ kipft/ft})) + (4 \times (-0,076433 \text{ kip/ft}) \times (6,5 \text{ ft}))}$$

$$a = 4,6631 \text{ ft}$$

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

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

$$V_{max} = ((-0.076433 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7854 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6631 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.7854 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6631 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.43601 \text{ kip}$$

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

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

$$M_{max} = ((-0.076433 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.7854 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6631 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7854 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6631 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7854 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6631 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2402 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{11.055 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.229 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.229 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = Minimum\ spacing\ of\ reinforcement,$</p> $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625\ in))]$ $s_{rebar} = 1.5\ in$ <p>Ties:</p> <p>Since longitudinal reinforcement is $\leq No. 10\emptyset$: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625\ in)), (48 \times (0.375\ in)), Min((48\ in), (48\ in))]$ $s_{ties} = 10\ in$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5\ ksi) \times [(2304\ in^2) - (4.2951\ in^2)]) + ((60\ ksi) \times (4.2951\ in^2))]$ $\phi P_N = 2675.2\ kip$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(11.055\ kip)}{(2675.2\ kip)}$ $Ratio = 0.0041324$	<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>$b_w = 48\ in$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48\ in)$ $d = 38.4\ in$ <p>λ_s - size effect modification factor</p> $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4\ in)}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ ksi \rightarrow 2500\ psi$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ psi)} \times (48\ in) \times (38.4\ in)$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.055 \text{ kip} \rightarrow 11055 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11055 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.96 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.96 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.96 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.96 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.05 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.237 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.237 \text{ kip})}{(111.05 \text{ kip})}$$

$$Ratio = 0.038153$$

Considering z-direction:

$V_{max} = 0.43601 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.43601 \text{ kip})}{(111.05 \text{ kip})}$$

$$Ratio = 0.0039261$$

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:

$M_{max} = 13.695 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.695 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.054868$$

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

Considering z-direction:

$M_{max} = 1.2402 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.2402 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0049686$$

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

REFERENCES	CALCULATIONS	RESULTS
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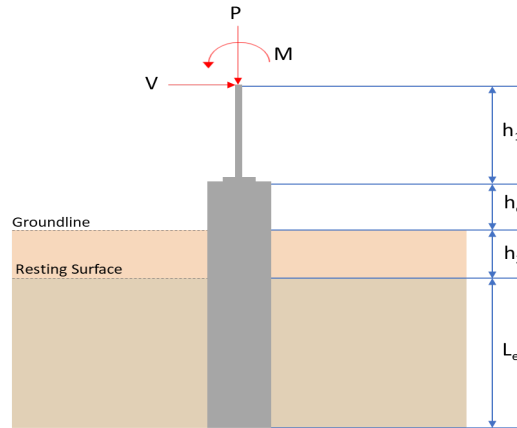
SkyCiv Foundation Design

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

$L = 6.75$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500,000	100,000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.734	15.102
V_x (kip)	-0.163	-0.272
V_z (kip)	-0.051	-0.081
M_x (kipft)	-0.142	-0.225
M_z (kipft)	16.416	27.884

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,

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

$$H_o = \frac{(-0.163 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.025955 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.416 \text{ kipft}) + ((-0.163 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.614 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.68 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.051 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.008121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.142 \text{ kipft}) + ((-0.051 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022611 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2211 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.68 \text{ ft}), (1.2211 \text{ ft})]$$

$$L_{e,req} = 6.68 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.68 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.98963$$

Status: **PASS**
Ratio: **0.990**

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

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

$$q = \frac{(9.734 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.60837 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.60837 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.40558$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.025955 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.614 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.614 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.614 \text{ kipft/ft})) + (4 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5241 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.614 \text{ kipft/ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.614 \text{ kipft/ft})) + (2 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.21668 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.614 \text{ kipft/ft})) + ((-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.66539 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.5241 \text{ ft})}{2}$$

$$p_a = 0.2262 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21668 \text{ kip/ft}^2)}{(0.2262 \text{ kip/ft}^2)}$$

$$Ratio = 0.95789$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.66539 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.98577$$

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

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = -0.008121 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.022611 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.022611 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.008121 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.022611 \text{ kipft/ft})) + (4 \times (-0.008121 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8475 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.022611 \text{ kipft/ft})) + (3 \times (-0.008121 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.022611 \text{ kipft/ft})) + (2 \times (-0.008121 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = -0.0021568 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.022611 \text{ kipft/ft})) + ((-0.008121 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = -0.0012634 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.8475 \text{ ft})}{2}$$

$$p_a = 0.24237 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0021568 \text{ kip/ft}^2)}{(0.24237 \text{ kip/ft}^2)}$$

$$Ratio = -0.0088985$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

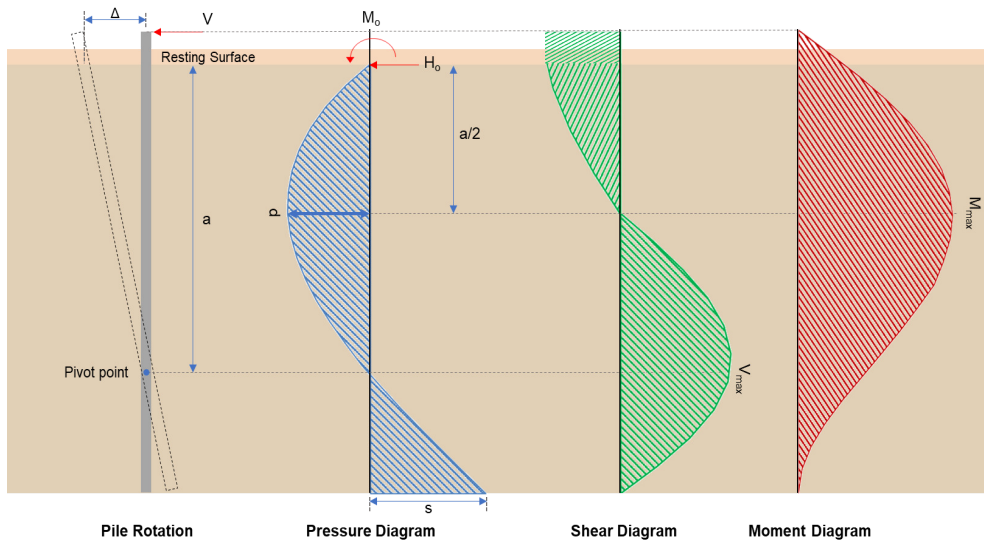
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0012634 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = -0.0018717$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.272 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.043312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.884 \text{ kipft}) + ((-0.272 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.4401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.4401 \text{ kipft/ft})}{(-0.043312 \text{ kip/ft})}$$

$$E = 102.51 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4401 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.4401 \text{ kipft/ft})) + (4 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(-0.043312 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.4401 \text{ kip/ft})) + (4 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5237 \text{ ft}$$

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

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

$$V_{max} = ((-0.043312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (102.51 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5237 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (102.51 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5237 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7872 \text{ kip}$$

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

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

$$M_{max} = ((-0.043312 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(102.51 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.5237 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (102.51 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5237 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (102.51 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5237 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 16.049 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.081 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.012898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.225 \text{ kipft}) + ((-0.081 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.035828 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.035828 \text{ kipft/ft})}{(-0.012898 \text{ kip/ft})}$$

$$E = 2.7778 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.035828 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.012898 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.035828 \text{ kipft/ft})) + (4 \times (-0.012898 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8478 \text{ ft}$$

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

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

$$V_{max} = ((-0.012898 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8478 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.7778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8478 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.072047 \text{ kip}$$

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

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

$$M_{max} = ((-0.012898 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(2.7778 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8478 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.7778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8478 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8478 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.21218 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{15.102 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.094 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.094 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(15.102 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0056452$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.102 \text{ kip} \rightarrow 15102 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15102 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.5 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.5 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.5 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.5 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.41 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.7872 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.7872 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.042971$$

Considering z-direction:

$V_{max} = 0.072047 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.072047 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.00064671$$

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:

$M_{max} = 16.049 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(16.049 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.064298$$

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

Considering z-direction:

$M_{max} = 0.21218 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.21218 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00085006$$

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

REFERENCES	CALCULATIONS	RESULTS
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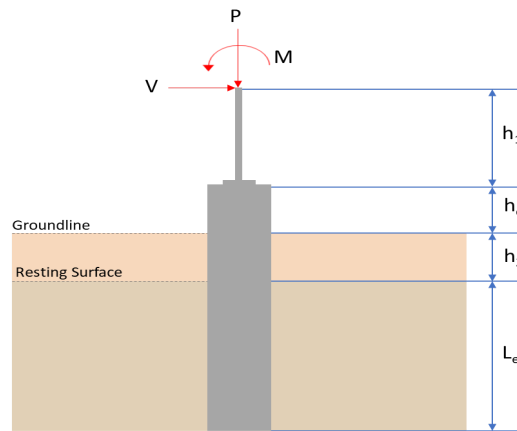
SkyCiv Foundation Design

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

$L = 6.75$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500,000	100,000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.735	15.105
V_x (kip)	-0.163	-0.272
V_z (kip)	0.052	0.083
M_x (kipft)	0.147	0.232
M_z (kipft)	16.418	27.888

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,

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

$$H_o = \frac{(-0.163 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.025955 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.418 \text{ kipft}) + ((-0.163 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6143 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.6803 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.052 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0082803 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.147 \text{ kipft}) + ((0.052 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.023408 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.5862 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.6803 \text{ ft}), (1.5862 \text{ ft})]$$

$$L_{e,req} = 6.68 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(6.68 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.98963$$

Status: **PASS**
Ratio: **0.990**

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

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

$$q = \frac{(9.735 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.60844 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.60844 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.40562$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.025955 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6143 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6143 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.6143 \text{ kipft/ft})) + (4 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5241 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6143 \text{ kipft/ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.6143 \text{ kipft/ft})) + (2 \times (-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.21671 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6143 \text{ kipft/ft})) + ((-0.025955 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.66548 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.5241 \text{ ft})}{2}$$

$$p_a = 0.2262 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21671 \text{ kip/ft}^2)}{(0.2262 \text{ kip/ft}^2)}$$

$$Ratio = 0.95802$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.66548 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.98589$$

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

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = 0.0082803 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.023408 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.023408 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.0082803 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.023408 \text{ kipft/ft})) + (4 \times (0.0082803 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8455 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.023408 \text{ kipft/ft})) + (3 \times (0.0082803 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.023408 \text{ kipft/ft})) + (2 \times (0.0082803 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.0061754 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.023408 \text{ kipft/ft})) + ((0.0082803 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.013525 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(4.8455 \text{ ft})}{2}$$

$$p_a = 0.24227 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0061754 \text{ kip/ft}^2)}{(0.24227 \text{ kip/ft}^2)}$$

$$Ratio = 0.025489$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

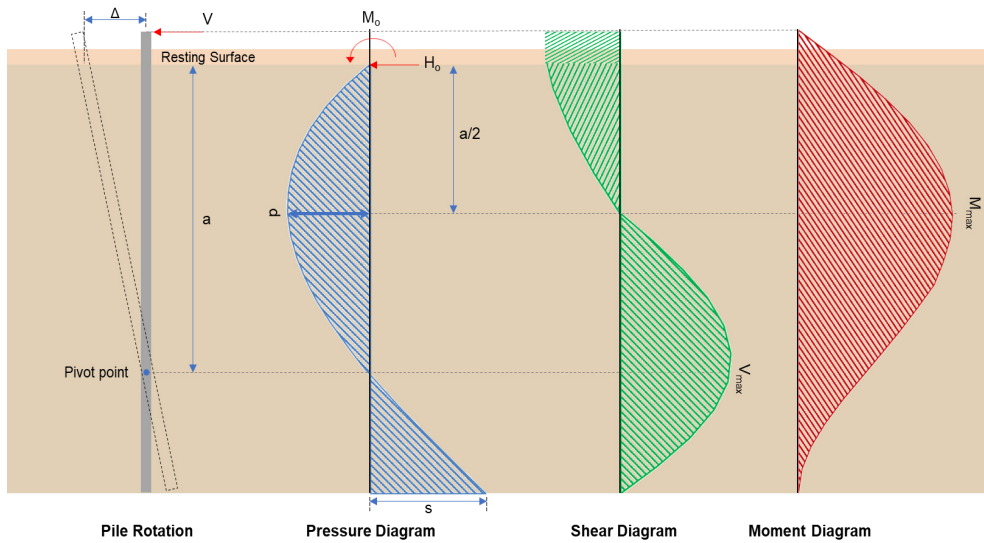
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.013525 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.020037$$

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,

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

$$H_o = \frac{(-0.272 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.043312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.888 \text{ kipft}) + ((-0.272 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.4408 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.4408 \text{ kipft/ft})}{(-0.043312 \text{ kip/ft})}$$

$$E = 102.53 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4408 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.4408 \text{ kipft/ft})) + (4 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(-0.043312 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.4408 \text{ kip/ft})) + (4 \times (-0.043312 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5236 \text{ ft}$$

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

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

$$V_{max} = ((-0.043312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (102.53 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5236 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (102.53 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5236 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7878 \text{ kip}$$

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

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

$$M_{max} = ((-0.043312 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(102.53 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.5236 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (102.53 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5236 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (102.53 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5236 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 16.051 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.083 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.013217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.232 \text{ kipft}) + ((0.083 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036943 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.036943 \text{ kipft/ft})}{(0.013217 \text{ kip/ft})}$$

$$E = 2.7952 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.036943 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.013217 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.036943 \text{ kipft/ft})) + (4 \times (0.013217 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.847 \text{ ft}$$

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

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

$$V_{max} = ((0.013217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7952 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.847 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.7952 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.847 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.074063 \text{ kip}$$

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

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

$$M_{max} = ((0.013217 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(2.7952 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.847 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.7952 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.847 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7952 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.847 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.21821 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{15.105 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.094 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.094 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</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>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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;">$Ratio = \frac{(15.105 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0056463$</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;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.105 \text{ kip} \rightarrow 15105 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15105 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.5 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.5 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.5 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.5 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.41 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.7878 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.7878 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.042977$$

Considering z-direction:

$V_{max} = 0.074063 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.074063 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.00066481$$

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:

$M_{max} = 16.051 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(16.051 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.064307$$

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

Considering z-direction:

$M_{max} = 0.21821 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.21821 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00087425$$

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