

Project Name: MTSOLAR_DBDKJ03GF8GG **Date:** Thu Oct 17 2024
Location: 1131 E Main St, Tustin, CA 92780, USA **Number of Modules:** 75
Unique ID: 5P-19.75-6TOP-SD-45-L-5Hx15W-J3CL **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	95.15 ft
Winter Tilt Angle	15
Front Edge Clearance	12 ft

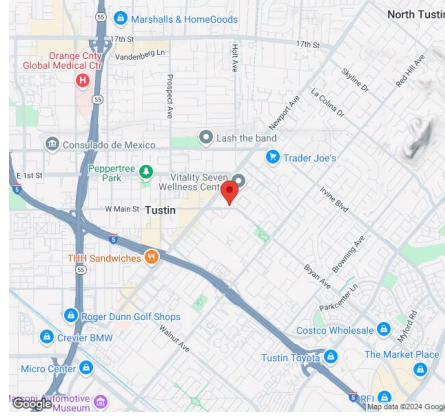
MT Solar Bill of Materials (5P-19.75-6TOP-SD-45-L-5Hx15W-J3CL)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-SD	H-Frame Assembly-SD	5
MTS-SD-Wing-45	45IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	8
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	15

Rail Bill of Materials

Part	Qty
Rails (223in)	30
Rail Attachment	120
Module Mid Clamp	120
Module End Clamp	60
Ground Lug	15

Site Details:



Site Address: 1131 E Main St, Tustin, CA 92780, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	75.12in
Number of Rows:	5
Number of Columns:	15
Total Number of Modules:	75
Winter Tilt Angle:	15
Front Edge Clearance:	12
Total Array Height at Tilt:	16.87 ft
Total Frame Length:	94.00 ft
Frame Weight:	5415 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	95.15 ft
Rail Length:	225.75 in
Rail Spacing:	3.17 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	14.43 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.00 ft Pile 3: 6.00 ft Pile 4: 6.00 ft Pile 5: 5.75 ft
Foundation Volume:	17.481 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1131 E Main St, Tustin, CA 92780, USA
Wind Speed:	90 mph

Snow Load:

0 psf

Design Disclaimer

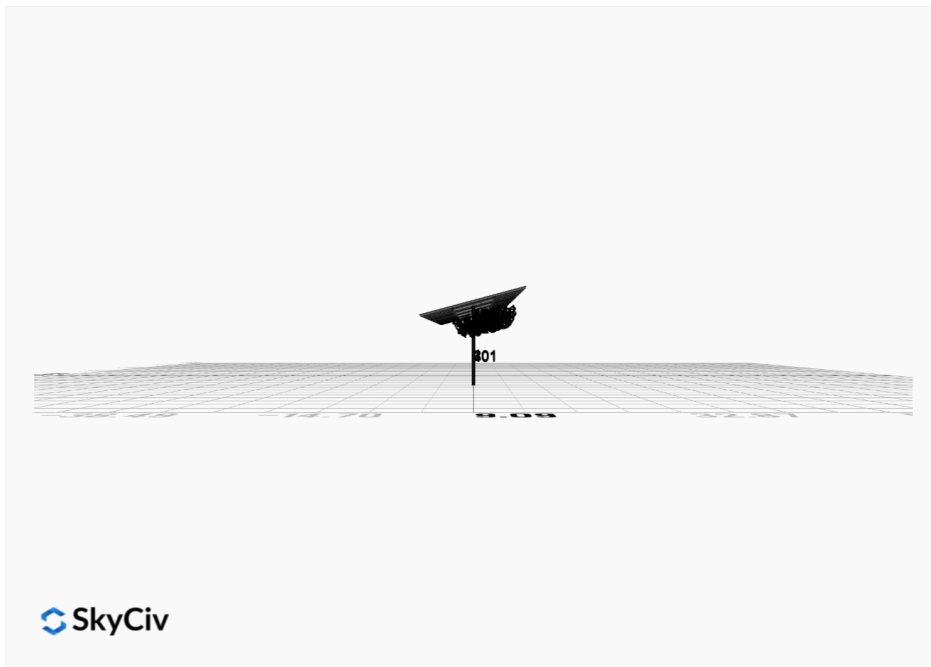
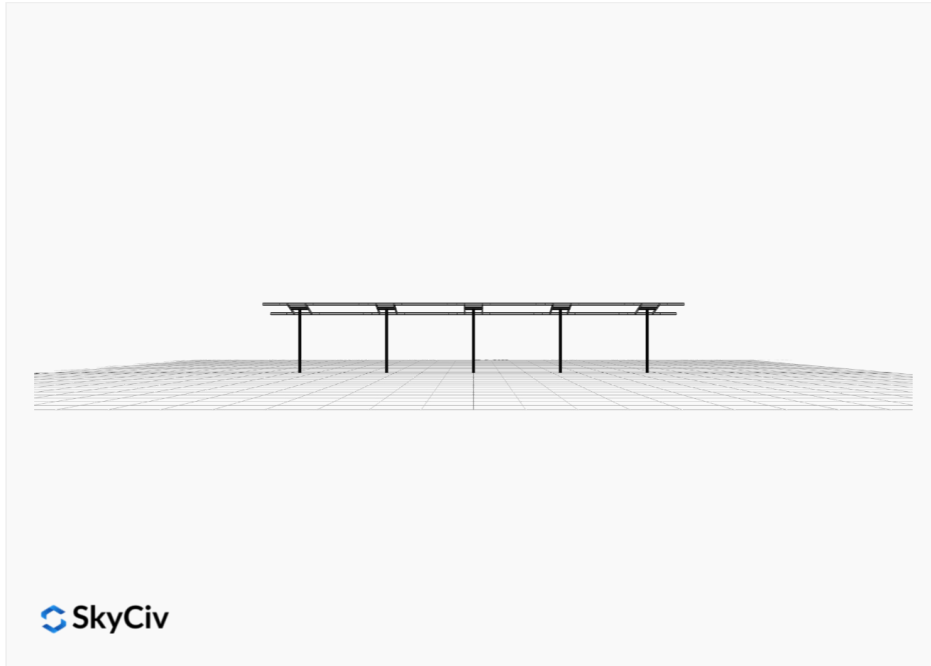
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

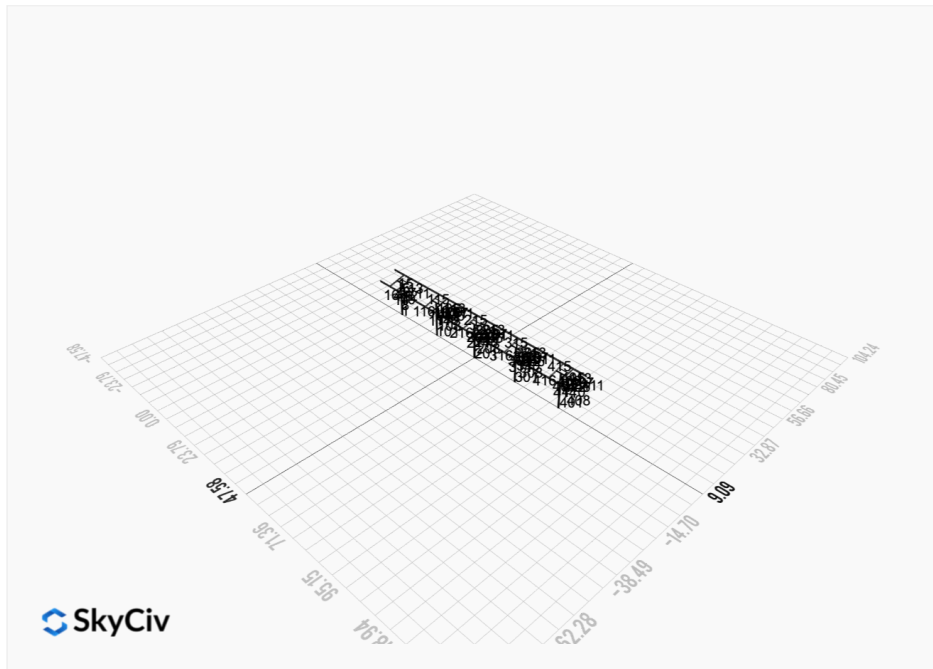
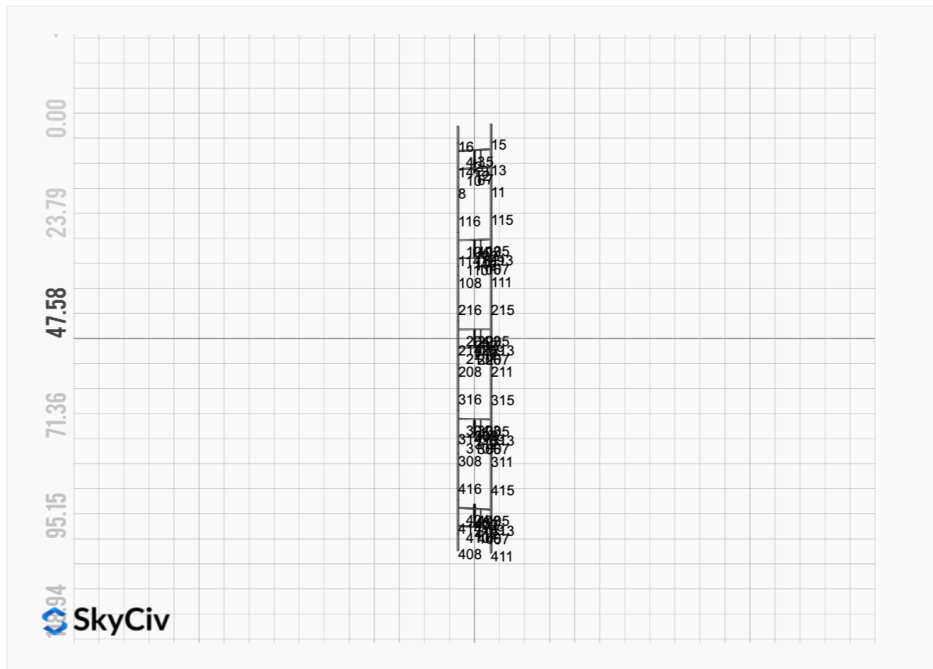
AutoDesigner Input

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Design Notes:

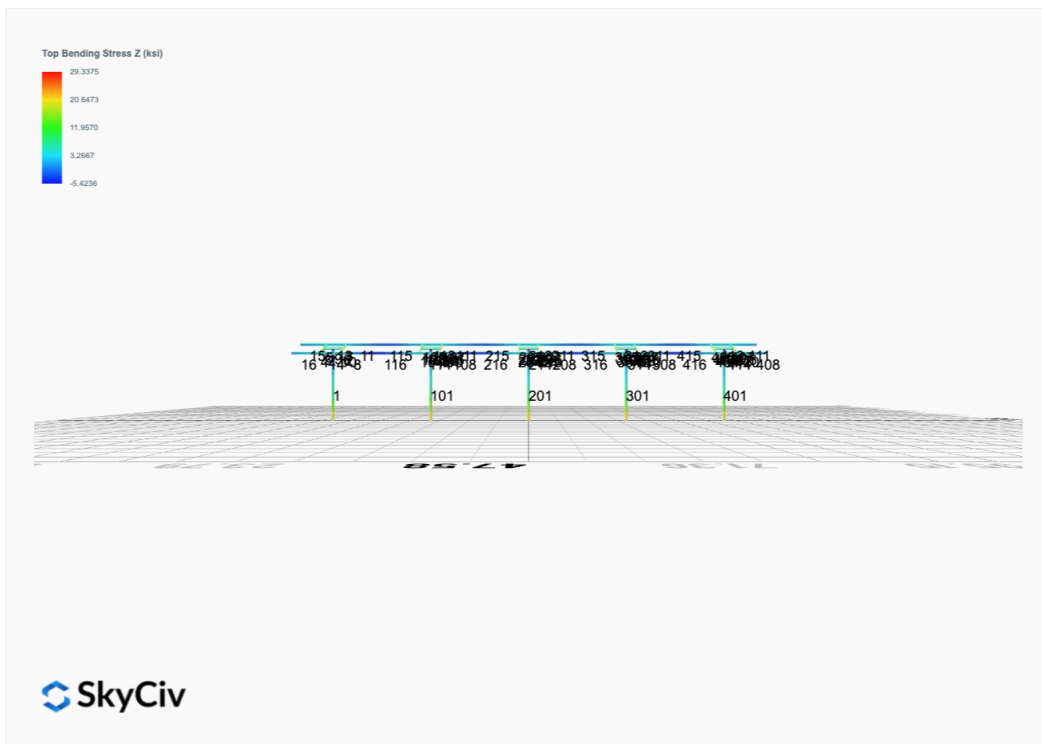
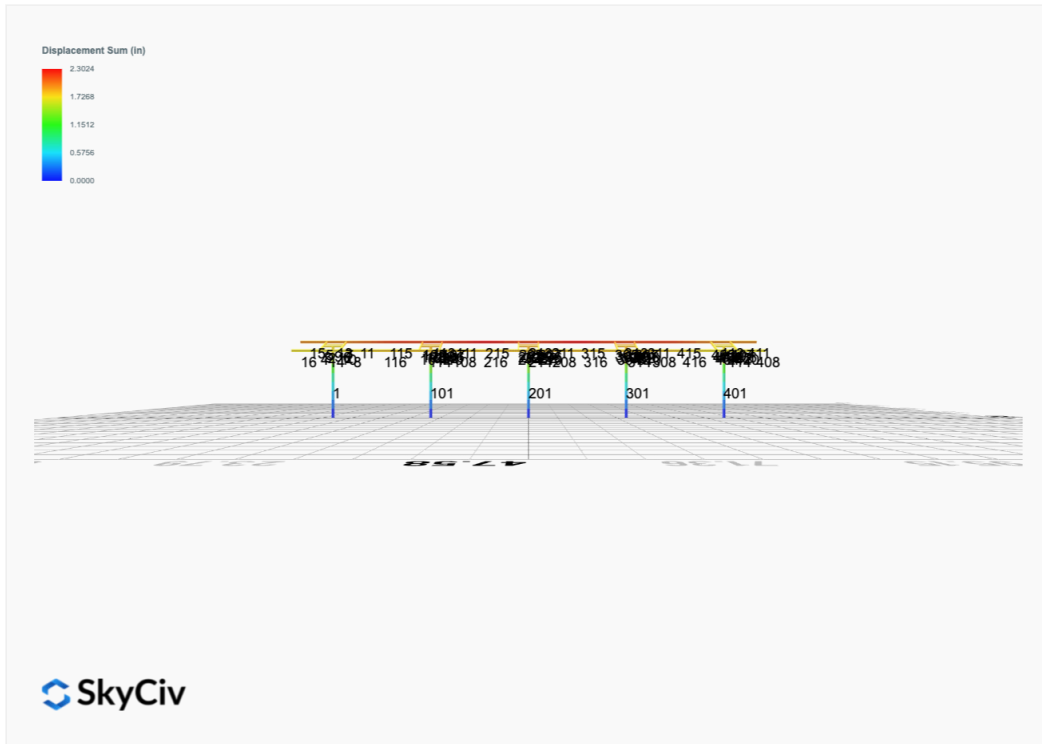
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

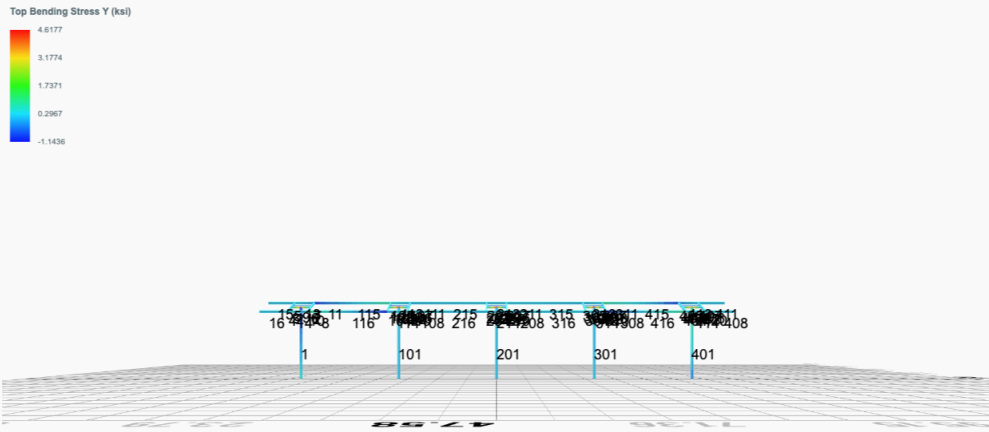




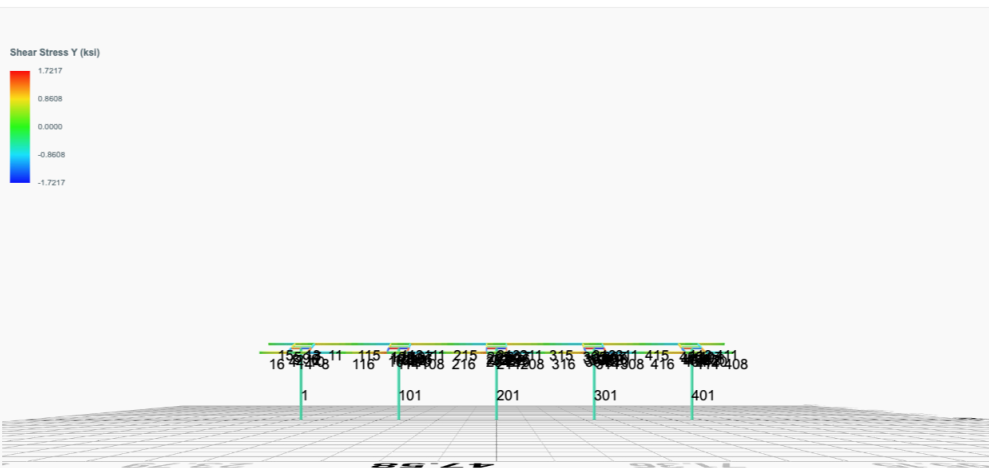


FEM Results (Envelope Worst Case for each member)

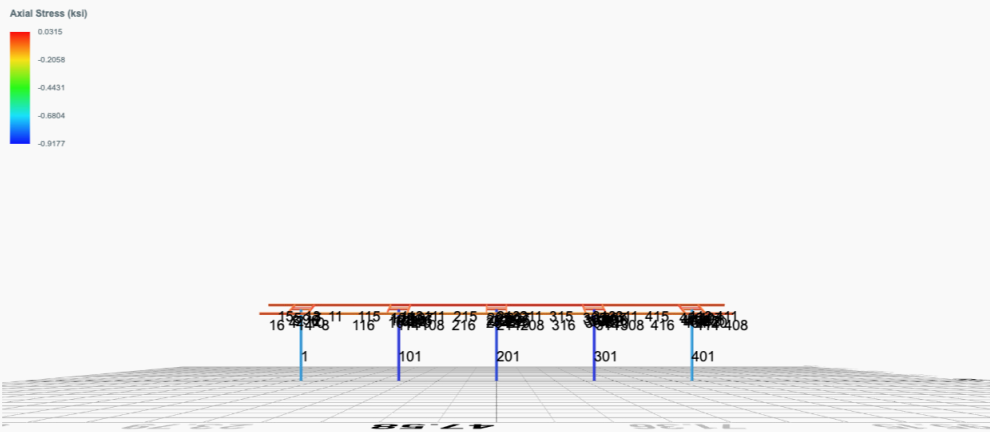




 SkyCiv



 SkyCiv



Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 2. D + L	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 3. D + (S or Lr or R)	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 3. D + (S or Lr or R)	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 5b. D + 0.7E	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0035	2.2070	0.0255	0.1195	-0.0086	-0.0236
ULS: 8. 0.6D + 0.7E	0.0021	1.3242	0.0153	0.0717	-0.0051	-0.0142
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9699	5.6842	0.0881	0.4104	-0.0863	16.7906
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.9699	5.6842	0.0881	0.4104	-0.0863	16.7906
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7265	-0.4295	-0.0169	-0.0757	0.0381	-8.2253
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6579	-0.0720	-0.0199	-0.0889	0.0548	-17.4788
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7266	4.8149	0.0725	0.3377	-0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7266	4.8149	0.0725	0.3377	-0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5458	0.2296	-0.0063	-0.0269	0.0265	-6.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4943	0.4977	-0.0085	-0.0368	0.0389	-13.1150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7266	4.8149	0.0725	0.3377	-0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7266	4.8149	0.0725	0.3377	-0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5458	0.2296	-0.0063	-0.0269	0.0265	-6.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4943	0.4977	-0.0085	-0.0368	0.0389	-13.1150
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9713	4.8014	0.0779	0.3626	-0.0829	16.8001
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.9713	4.8014	0.0779	0.3626	-0.0829	16.8001
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7251	-1.3123	-0.0271	-0.1235	0.0416	-8.2159
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6565	-0.9548	-0.0301	-0.1367	0.0582	-17.4694

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	8.4440
Shear X	-1.6232
Shear Z	0.1366
Moment X	0.6381
Moment Y (Twist)	0.1437
Moment Z	30.3559

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	5.6842
Shear X	-0.9713
Shear Z	0.0881
Moment X	0.4104
Moment Y (Twist)	0.0863
Moment Z	17.4788

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	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 2. D + L	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 3. D + (S or Lr or R)	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 3. D + (S or Lr or R)	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 5b. D + 0.7E	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0035	2.5008	-0.0029	-0.0139	0.0019	0.0656
ULS: 8. 0.6D + 0.7E	-0.0021	1.5005	-0.0018	-0.0083	0.0011	0.0394
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0533	6.5479	-0.0017	-0.0100	-0.0230	18.3366
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0533	6.5479	-0.0017	-0.0100	-0.0230	18.3366
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8050	-0.5724	-0.0006	-0.0022	0.0088	-8.9329
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6716	-0.1480	-0.0062	-0.0273	0.0304	-18.5079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7908	5.5361	-0.0020	-0.0110	-0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7908	5.5361	-0.0020	-0.0110	-0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6029	0.1959	-0.0012	-0.0051	0.0071	-6.6832
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5028	0.5142	-0.0054	-0.0240	0.0233	-13.8645
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7908	5.5361	-0.0020	-0.0110	-0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7908	5.5361	-0.0020	-0.0110	-0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6029	0.1959	-0.0012	-0.0051	0.0071	-6.6832
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5028	0.5142	-0.0054	-0.0240	0.0233	-13.8645
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0519	5.5476	-0.0005	-0.0045	-0.0238	18.3104
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0519	5.5476	-0.0005	-0.0045	-0.0238	18.3104
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8064	-1.5727	0.0006	0.0034	0.0080	-8.9591
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6730	-1.1483	-0.0051	-0.0218	0.0297	-18.5341

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	9.7453
Shear X	-1.7499
Shear Z	-0.0098
Moment X	-0.0426
Moment Y (Twist)	0.0533
Moment Z	32.2279

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.5479
Shear X	-1.0533
Shear Z	-0.0062
Moment X	-0.0273
Moment Y (Twist)	0.0304
Moment Z	18.5341

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	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 2. D + L	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 3. D + (S or Lr or R)	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 3. D + (S or Lr or R)	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 5b. D + 0.7E	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.4705	-0.0000	0.0000	0.0000	0.0306
ULS: 8. 0.6D + 0.7E	0.0000	1.4823	-0.0000	0.0000	0.0000	0.0183
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0572	6.4691	-0.0000	0.0000	0.0000	18.5141
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0572	6.4691	-0.0000	0.0000	0.0000	18.5141
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8087	-0.5598	0.0000	0.0000	0.0000	-9.0107
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6849	-0.1531	-0.0000	0.0000	0.0000	-18.8116

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	-0.7929	5.4694	-0.0000	0.0000	0.0000	13.8932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7929	5.4694	-0.0000	0.0000	0.0000	13.8932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6065	0.1978	-0.0000	0.0000	0.0000	-6.7504
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5137	0.5028	-0.0000	0.0000	0.0000	-14.1010
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7929	5.4694	-0.0000	0.0000	0.0000	13.8932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7929	5.4694	-0.0000	0.0000	0.0000	13.8932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6065	0.1978	-0.0000	0.0000	0.0000	-6.7504
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5137	0.5028	-0.0000	0.0000	0.0000	-14.1010
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0572	5.4809	-0.0000	-0.0000	0.0000	18.5019
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0572	5.4809	-0.0000	-0.0000	0.0000	18.5019
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8087	-1.5480	0.0000	-0.0000	0.0000	-9.0230
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6849	-1.1413	-0.0000	0.0000	0.0000	-18.8238

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	9.6299
Shear X	-1.7620
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0000
Moment Z	32.7527

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.4691
Shear X	-1.0572
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	18.8238

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	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 2. D + L	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 3. D + (S or Lr or R)	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 3. D + (S or Lr or R)	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 5b. D + 0.7E	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0035	2.5008	0.0029	0.0139	-0.0019	0.0656
ULS: 8. 0.6D + 0.7E	-0.0021	1.5005	0.0018	0.0083	-0.0011	0.0394
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0533	6.5479	0.0017	0.0100	0.0230	18.3366
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0533	6.5479	0.0017	0.0100	0.0230	18.3366
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8050	-0.5724	0.0006	0.0022	-0.0088	-8.9329
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6716	-0.1480	0.0062	0.0274	-0.0304	-18.5079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7908	5.5361	0.0020	0.0110	0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7908	5.5361	0.0020	0.0110	0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6029	0.1959	0.0012	0.0051	-0.0071	-6.6832
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5028	0.5142	0.0054	0.0240	-0.0233	-13.8645
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7908	5.5361	0.0020	0.0110	0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7908	5.5361	0.0020	0.0110	0.0168	13.7689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6029	0.1959	0.0012	0.0051	-0.0071	-6.6832
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5028	0.5142	0.0054	0.0240	-0.0233	-13.8645

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0519	5.5476	0.0005	0.0045	0.0238	18.3104
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0519	5.5476	0.0005	0.0045	0.0238	18.3104
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8064	-1.5727	-0.0006	-0.0034	-0.0080	-8.9591
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6730	-1.1483	0.0051	0.0218	-0.0297	-18.5341

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	9.7453
Shear X	-1.7499
Shear Z	0.0098
Moment X	0.0427
Moment Y (Twist)	0.0532
Moment Z	32.2280

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.5479
Shear X	-1.0533
Shear Z	0.0062
Moment X	0.0274
Moment Y (Twist)	0.0304
Moment Z	18.5341

Reaction Forces for Foundation 5 (Node ID#401), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 2. D + L	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 3. D + (S or Lr or R)	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 3. D + (S or Lr or R)	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 5b. D + 0.7E	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0035	2.2070	-0.0255	-0.1195	0.0086	-0.0236
ULS: 8. 0.6D + 0.7E	0.0021	1.3242	-0.0153	-0.0717	0.0051	-0.0142
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9699	5.6842	-0.0881	-0.4104	0.0863	16.7906
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.9699	5.6842	-0.0881	-0.4104	0.0863	16.7906
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7265	-0.4295	0.0169	0.0757	-0.0381	-8.2253
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6579	-0.0720	0.0199	0.0889	-0.0548	-17.4788
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7266	4.8149	-0.0725	-0.3377	0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7266	4.8149	-0.0725	-0.3377	0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5458	0.2296	0.0063	0.0269	-0.0265	-6.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4943	0.4977	0.0085	0.0368	-0.0389	-13.1150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7266	4.8149	-0.0725	-0.3377	0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7266	4.8149	-0.0725	-0.3377	0.0668	12.5871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5458	0.2296	0.0063	0.0269	-0.0265	-6.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4943	0.4977	0.0085	0.0368	-0.0389	-13.1150
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9713	4.8014	-0.0779	-0.3626	0.0829	16.8001
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.9713	4.8014	-0.0779	-0.3626	0.0829	16.8001
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7251	-1.3123	0.0271	0.1235	-0.0416	-8.2159
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6565	-0.9548	0.0301	0.1367	-0.0582	-17.4694

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.

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.4440
Shear X	-1.6231
Shear Z	-0.1366
Moment X	-0.6381
Moment Y (Twist)	0.1437
Moment Z	30.3563

Result	Value (kip, kip-ft)
Axial	5.6842
Shear X	-0.9713
Shear Z	-0.0881
Moment X	-0.4104
Moment Y (Twist)	0.0863
Moment Z	17.4788

Project Details

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

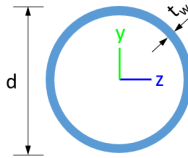


Design Input Information

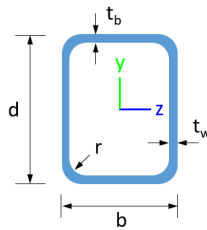
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

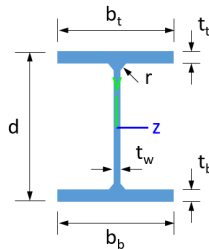
Section Dimensions



ID	Name	d (in)	t _w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
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104	79.65	72.01	10.99	6.26	29.14	16.61
105	79.65	73.44	10.99	6.26	29.14	16.61
106	79.65	74.02	10.99	6.26	29.14	16.61
107	79.65	73.44	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.34	6.45	30.09	45.74
114	120.60	84.03	18.31	6.45	30.09	45.74
115	120.60	68.63	15.29	6.45	30.09	45.74
116	120.60	68.63	15.24	6.45	30.09	45.74
201	251.16	48.05	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	6.26	29.14	16.61
204	79.65	72.01	10.99	6.26	29.14	16.61
205	79.65	73.44	10.99	6.26	29.14	16.61
206	79.65	74.02	10.99	6.26	29.14	16.61
207	79.65	73.44	10.99	6.26	29.14	16.61
208	120.60	115.40	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	6.26	29.14	16.61
211	120.60	115.40	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.03	18.27	6.45	30.09	45.74
214	120.60	84.03	18.25	6.45	30.09	45.74
215	120.60	68.63	15.66	6.45	30.09	45.74
216	120.60	68.63	15.76	6.45	30.09	45.74
301	251.16	48.05	42.30	42.30	75.35	75.35
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	6.26	29.14	16.61
304	79.65	72.01	10.99	6.26	29.14	16.61
305	79.65	73.44	10.99	6.26	29.14	16.61
306	79.65	74.02	10.99	6.26	29.14	16.61
307	79.65	73.44	10.99	6.26	29.14	16.61
308	120.60	115.40	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	6.26	29.14	16.61
311	120.60	115.40	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	84.03	18.34	6.45	30.09	45.74
314	120.60	84.03	18.31	6.45	30.09	45.74
315	120.60	68.63	15.62	6.45	30.09	45.74
316	120.60	68.63	15.69	6.45	30.09	45.74
401	251.16	48.05	42.30	42.30	75.35	75.35
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.02	10.99	6.26	29.14	16.61
404	79.65	72.01	10.99	6.26	29.14	16.61
405	79.65	73.44	10.99	6.26	29.14	16.61
406	79.65	74.02	10.99	6.26	29.14	16.61
407	79.65	73.44	10.99	6.26	29.14	16.61

407	79.03	73.44	10.99	6.20	29.14	10.01
408	120.60	54.44	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.01	10.99	6.26	29.14	16.61
411	120.60	54.44	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	84.03	19.13	6.45	30.09	45.74
414	120.60	84.03	19.47	6.45	30.09	45.74
415	120.60	68.63	15.28	6.45	30.09	45.74
416	120.60	68.63	15.24	6.45	30.09	45.74

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	0.176	0.718	0.031	0.022	0.002	0.793	#13	0.810	Not Required	Pass
2	0.001	0.409	0.095	0.089	0.017	0.504	#13	0.034	Not Required	Pass
3	0.002	0.706	0.015	0.071	0.005	0.722	#13	0.044	Not Required	Pass
4	0.002	0.605	0.038	0.061	0.007	0.642	#13	0.078	Not Required	Pass
5	0.002	0.437	0.031	0.071	0.007	0.440	#13	0.073	Not Required	Pass
6	0.003	0.804	0.024	0.082	0.004	0.824	#13	0.044	Not Required	Pass
7	0.003	0.499	0.070	0.081	0.014	0.504	#13	0.073	Not Required	Pass
8	0.001	0.083	0.052	0.042	0.003	0.116	#13	0.088	Not Required	Pass
9	0.004	0.091	0.023	0.002	0.001	0.114	#13	0.198	Not Required	Pass
10	0.003	0.673	0.074	0.068	0.013	0.682	#13	0.078	Not Required	Pass
11	0.001	0.088	0.052	0.051	0.003	0.119	#13	0.088	Not Required	Pass
12	0.001	0.491	0.104	0.101	0.021	0.595	#13	0.052	Not Required	Pass
13	0.001	0.216	0.093	0.066	0.004	0.236	#13	0.265	Not Required	Pass
14	0.002	0.185	0.093	0.055	0.004	0.206	#13	0.177	Not Required	Pass
15	0.000	0.076	0.021	0.032	0.002	0.094	#13	Not Required	Not Required	Pass
16	0.000	0.065	0.021	0.027	0.002	0.083	#13	Not Required	Not Required	Pass
101	0.203	0.762	0.002	0.023	0.000	0.873	#13	0.810	Not Required	Pass
102	0.001	0.522	0.110	0.110	0.019	0.632	#13	0.052	Not Required	Pass
103	0.003	0.862	0.025	0.087	0.004	0.889	#13	0.044	Not Required	Pass
104	0.003	0.755	0.069	0.076	0.011	0.801	#13	0.078	Not Required	Pass
105	0.003	0.534	0.071	0.086	0.014	0.551	#13	0.073	Not Required	Pass
106	0.003	0.870	0.028	0.088	0.007	0.876	#13	0.044	Not Required	Pass
107	0.003	0.540	0.044	0.087	0.009	0.552	#13	0.073	Not Required	Pass
108	0.002	0.055	0.026	0.044	0.003	0.068	#13	0.088	Not Required	Pass
109	0.004	0.089	0.034	0.001	0.001	0.118	#13	0.198	Not Required	Pass
110	0.002	0.745	0.044	0.075	0.008	0.765	#13	0.078	Not Required	Pass
111	0.002	0.069	0.026	0.052	0.003	0.081	#13	0.088	Not Required	Pass
112	0.001	0.522	0.113	0.109	0.022	0.635	#13	0.052	Not Required	Pass
113	0.003	0.238	0.095	0.067	0.004	0.308	#13	0.265	Not Required	Pass
114	0.004	0.224	0.095	0.059	0.004	0.299	#13	0.265	Not Required	Pass
115	0.002	0.258	0.054	0.052	0.003	0.290	#13	0.439	Not Required	Pass
116	0.002	0.222	0.054	0.046	0.003	0.252	#13	0.439	Not Required	Pass
201	0.200	0.774	0.000	0.023	0.000	0.878	#13	0.810	Not Required	Pass
202	0.001	0.516	0.111	0.109	0.021	0.627	#13	0.034	Not Required	Pass
203	0.003	0.861	0.020	0.087	0.003	0.879	#13	0.044	Not Required	Pass
204	0.002	0.736	0.041	0.074	0.007	0.765	#13	0.078	Not Required	Pass
205	0.003	0.534	0.043	0.086	0.008	0.545	#13	0.073	Not Required	Pass

206	0.003	0.861	0.020	0.087	0.003	0.879	#13	0.044	Not Required	Pass
207	0.003	0.534	0.043	0.086	0.008	0.545	#13	0.073	Not Required	Pass
208	0.002	0.057	0.024	0.044	0.003	0.065	#13	0.088	Not Required	Pass
209	0.003	0.085	0.025	0.001	0.000	0.111	#13	0.198	Not Required	Pass
210	0.002	0.736	0.041	0.074	0.007	0.765	#13	0.078	Not Required	Pass
211	0.002	0.070	0.024	0.051	0.003	0.074	#13	0.088	Not Required	Pass
212	0.001	0.516	0.111	0.109	0.021	0.627	#13	0.034	Not Required	Pass
213	0.003	0.239	0.057	0.066	0.003	0.281	#13	0.265	Not Required	Pass
214	0.004	0.216	0.056	0.057	0.003	0.254	#13	0.265	Not Required	Pass
215	0.003	0.235	0.033	0.051	0.003	0.265	#13	0.439	Not Required	Pass
216	0.004	0.197	0.033	0.044	0.003	0.228	#13	0.439	Not Required	Pass
301	0.203	0.762	0.002	0.023	0.000	0.873	#13	0.810	Not Required	Pass
302	0.001	0.522	0.113	0.109	0.022	0.635	#13	0.052	Not Required	Pass
303	0.003	0.870	0.028	0.088	0.007	0.876	#13	0.044	Not Required	Pass
304	0.002	0.745	0.044	0.075	0.008	0.765	#13	0.078	Not Required	Pass
305	0.003	0.540	0.044	0.087	0.009	0.552	#13	0.073	Not Required	Pass
306	0.003	0.862	0.025	0.087	0.004	0.889	#13	0.044	Not Required	Pass
307	0.003	0.534	0.071	0.086	0.014	0.551	#13	0.073	Not Required	Pass
308	0.001	0.062	0.054	0.046	0.003	0.096	#13	0.088	Not Required	Pass
309	0.004	0.089	0.034	0.001	0.001	0.118	#13	0.198	Not Required	Pass
310	0.003	0.755	0.069	0.076	0.011	0.801	#13	0.078	Not Required	Pass
311	0.001	0.081	0.054	0.052	0.003	0.097	#13	0.088	Not Required	Pass
312	0.001	0.522	0.110	0.110	0.019	0.632	#13	0.052	Not Required	Pass
313	0.003	0.238	0.095	0.067	0.004	0.308	#13	0.265	Not Required	Pass
314	0.004	0.224	0.095	0.059	0.004	0.299	#13	0.265	Not Required	Pass
315	0.003	0.235	0.033	0.052	0.003	0.264	#13	0.439	Not Required	Pass
316	0.004	0.196	0.033	0.044	0.003	0.226	#13	0.439	Not Required	Pass
401	0.176	0.718	0.031	0.022	0.002	0.793	#13	0.810	Not Required	Pass
402	0.001	0.491	0.104	0.101	0.021	0.595	#13	0.052	Not Required	Pass
403	0.003	0.804	0.024	0.082	0.004	0.824	#13	0.044	Not Required	Pass
404	0.003	0.673	0.074	0.068	0.013	0.682	#13	0.078	Not Required	Pass
405	0.003	0.499	0.070	0.081	0.014	0.504	#13	0.073	Not Required	Pass
406	0.002	0.706	0.015	0.071	0.005	0.722	#13	0.044	Not Required	Pass
407	0.002	0.437	0.031	0.071	0.007	0.440	#13	0.073	Not Required	Pass
408	0.000	0.065	0.021	0.027	0.002	0.083	#13	Not Required	Not Required	Pass
409	0.004	0.091	0.023	0.002	0.001	0.114	#13	0.198	Not Required	Pass
410	0.002	0.605	0.038	0.061	0.007	0.642	#13	0.078	Not Required	Pass
411	0.000	0.076	0.021	0.032	0.002	0.094	#13	Not Required	Not Required	Pass
412	0.001	0.409	0.095	0.089	0.017	0.504	#13	0.034	Not Required	Pass
413	0.001	0.216	0.093	0.066	0.004	0.236	#13	0.177	Not Required	Pass
414	0.002	0.185	0.093	0.055	0.004	0.206	#13	0.265	Not Required	Pass
415	0.002	0.265	0.052	0.051	0.003	0.289	#13	0.439	Not Required	Pass
416	0.002	0.227	0.052	0.042	0.003	0.261	#13	0.439	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength

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

REFERENCES	CALCULATIONS	RESULTS
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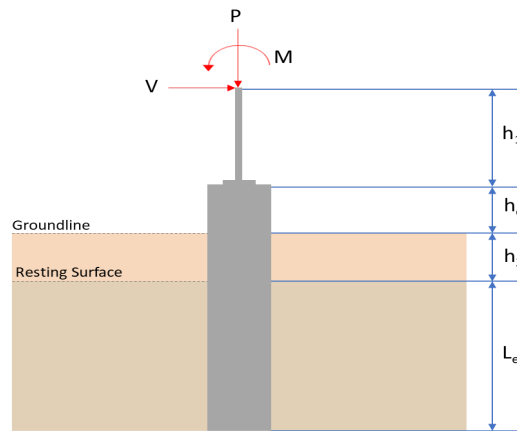
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.684	8.444
V_x (kip)	-0.971	-1.623
V_z (kip)	0.088	0.137
M_x (kipft)	0.410	0.638
M_z (kipft)	17.479	30.356

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,

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

$$H_o = \frac{(-0.971 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15462 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.479 \text{ kipft}) + ((-0.971 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7833 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5522 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.088 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.014013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.41 \text{ kipft}) + ((0.088 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.065287 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8961 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5522 \text{ ft}), (1.8961 \text{ ft})]$$

$$L_{e,req} = 5.552 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.552 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.96557$$

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

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

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

$$q = \frac{(5.684 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.35525 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35525 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17763$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.15462 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7833 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.7833 \text{ kipft/ft})) + (4 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9175 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.7833 \text{ kipft/ft})) + (3 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.7833 \text{ kipft/ft})) + (2 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.2474 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.7833 \text{ kipft/ft})) + ((-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.84885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9175 \text{ ft})}{2}$$

$$p_a = 0.29381 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2474 \text{ kip/ft}^2)}{(0.29381 \text{ kip/ft}^2)}$$

$$Ratio = 0.84204$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.84885 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.98417$$

Status: **PASS**
Ratio: **0.840**

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

Considering z-direction:

$H_o = 0.014013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.065287 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.065287 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.014013 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.065287 \text{ kipft/ft})) + (4 \times (0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0496 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.065287 \text{ kipft/ft})) + (3 \times (0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.065287 \text{ kipft/ft})) + (2 \times (0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.016068 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.065287 \text{ kipft/ft})) + ((0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.038318 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0496 \text{ ft})}{2}$$

$$p_a = 0.30372 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.016068 \text{ kip/ft}^2)}{(0.30372 \text{ kip/ft}^2)}$$

$$Ratio = 0.052903$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

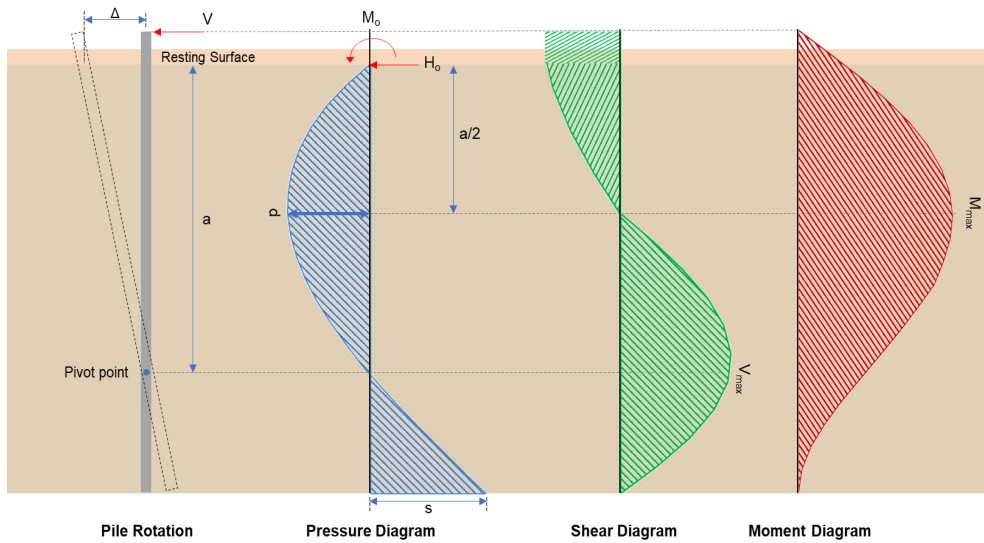
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.038318 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.044426$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.623 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25844 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.356 \text{ kipft}) + ((-1.623 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.8338 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.8338 \text{ kipft/ft})}{(-0.25844 \text{ kip/ft})}$$

$$E = 18.704 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8338 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.8338 \text{ kipft/ft})) + (4 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.25844 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.8338 \text{ kipft/ft})) + (4 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9148 \text{ ft}$$

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

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

$$V_{max} = ((-0.25844 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9148 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9148 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.6387 \text{ kip}$$

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

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

$$M_{max} = ((-0.25844 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(18.704 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.543 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.137 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.021815 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.638 \text{ kipft}) + ((0.137 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10159 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10159 \text{ kipft/ft})}{(0.021815 \text{ kip/ft})}$$

$$E = 4.6569 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10159 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.021815 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.10159 \text{ kipft/ft})) + (4 \times (0.021815 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0497 \text{ ft}$$

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

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

$$V_{max} = ((0.021815 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0497 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0497 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.18281 \text{ kip}$$

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

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

$$M_{max} = ((0.021815 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.6569 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.48052 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.444 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.316 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.316 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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;">$Ratio = \frac{(8.444 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0031564$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.444 \text{ kip} \rightarrow 8444 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8444 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.61 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.61 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.61 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.61 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.83 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6387 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.6387 \text{ kip})}{(110.83 \text{ kip})}$$

$$Ratio = 0.059901$$

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

Considering z-direction:

$V_{max} = 0.18281 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.18281 \text{ kip})}{(110.83 \text{ kip})}$$

$$Ratio = 0.0016495$$

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:

$M_{max} = 18.543 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.543 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.074289$$

Status: **PASS**
Ratio: **0.070**

Considering z-direction:

$M_{max} = 0.48052 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.48052 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0019252$$

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

REFERENCES	CALCULATIONS	RESULTS
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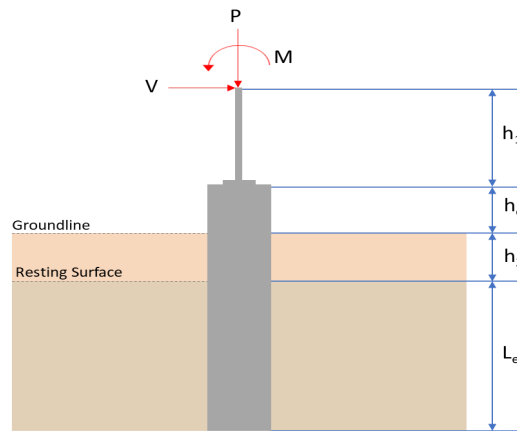
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.684	8.444
V_x (kip)	-0.971	-1.623
V_z (kip)	-0.088	-0.137
M_x (kipft)	-0.410	-0.638
M_z (kipft)	17.479	30.356

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,

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

$$H_o = \frac{(-0.971 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15462 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.479 \text{ kipft}) + ((-0.971 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7833 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5522 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.088 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.014013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.41 \text{ kipft}) + ((-0.088 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.065287 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5741 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5522 \text{ ft}), (1.5741 \text{ ft})]$$

$$L_{e,req} = 5.552 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.552 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.96557$$

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

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

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

$$q = \frac{(5.684 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.35525 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35525 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17763$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.15462 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7833 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.7833 \text{ kipft/ft})) + (4 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9175 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.7833 \text{ kipft/ft})) + (3 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.7833 \text{ kipft/ft})) + (2 \times (-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.2474 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.7833 \text{ kipft/ft})) + ((-0.15462 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.84885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9175 \text{ ft})}{2}$$

$$p_a = 0.29381 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2474 \text{ kip/ft}^2)}{(0.29381 \text{ kip/ft}^2)}$$

$$Ratio = 0.84204$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.84885 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.98417$$

Status: **PASS**
Ratio: **0.840**

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

Considering z-direction:

$H_o = -0.014013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.065287 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.065287 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.014013 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.065287 \text{ kipft/ft})) + (4 \times (-0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0496 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.065287 \text{ kipft/ft})) + (3 \times (-0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.065287 \text{ kipft/ft})) + (2 \times (-0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.00024662 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.065287 \text{ kipft/ft})) + ((-0.014013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.0090737 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0496 \text{ ft})}{2}$$

$$p_a = 0.30372 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00024662 \text{ kip/ft}^2)}{(0.30372 \text{ kip/ft}^2)}$$

$$Ratio = 0.000812$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

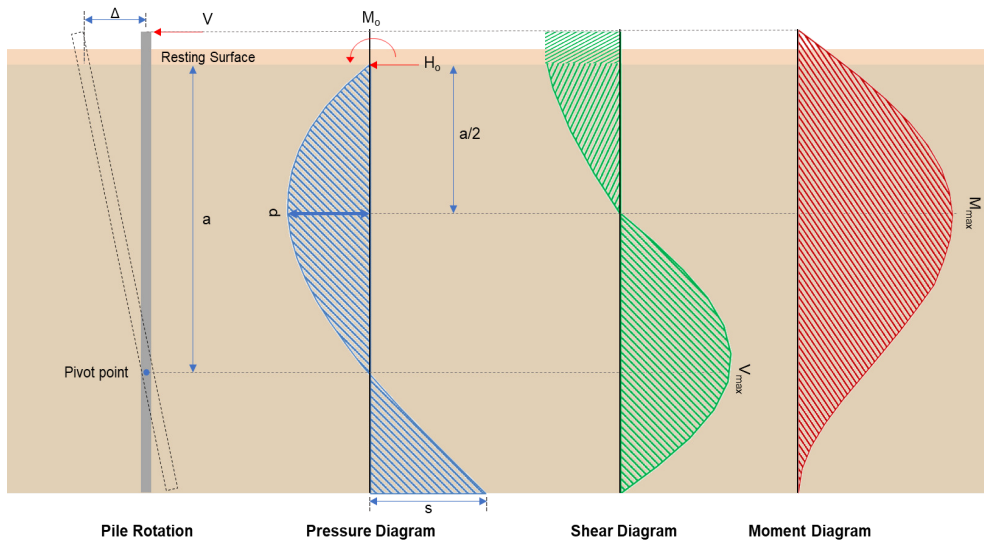
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0090737 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.01052$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.623 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25844 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.356 \text{ kipft}) + ((-1.623 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.8338 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.8338 \text{ kipft/ft})}{(-0.25844 \text{ kip/ft})}$$

$$E = 18.704 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8338 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.8338 \text{ kipft/ft})) + (4 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.25844 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.8338 \text{ kipft/ft})) + (4 \times (-0.25844 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9148 \text{ ft}$$

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

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

$$V_{max} = ((-0.25844 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9148 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9148 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.6387 \text{ kip}$$

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

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

$$M_{max} = ((-0.25844 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(18.704 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.704 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9148 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.543 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.137 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.021815 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.638 \text{ kipft}) + ((-0.137 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10159 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10159 \text{ kipft/ft})}{(-0.021815 \text{ kip/ft})}$$

$$E = 4.6569 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10159 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.021815 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.10159 \text{ kipft/ft})) + (4 \times (-0.021815 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0497 \text{ ft}$$

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

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

$$V_{max} = ((-0.021815 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0497 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0497 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.18281 \text{ kip}$$

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

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

$$M_{max} = ((-0.021815 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.6569 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.6569 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0497 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.48052 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.444 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.316 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.316 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.444 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0031564$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.444 \text{ kip} \rightarrow 8444 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8444 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.61 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.61 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.61 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.61 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.83 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6387 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.6387 \text{ kip})}{(110.83 \text{ kip})}$$

$$Ratio = 0.059901$$

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

Considering z-direction:

$V_{max} = 0.18281 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.18281 \text{ kip})}{(110.83 \text{ kip})}$$

$$Ratio = 0.0016495$$

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:

$M_{max} = 18.543 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.543 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.074289$$

Status: **PASS**
Ratio: **0.070**

Considering z-direction:

$M_{max} = 0.48052 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.48052 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0019252$$

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

REFERENCES	CALCULATIONS	RESULTS
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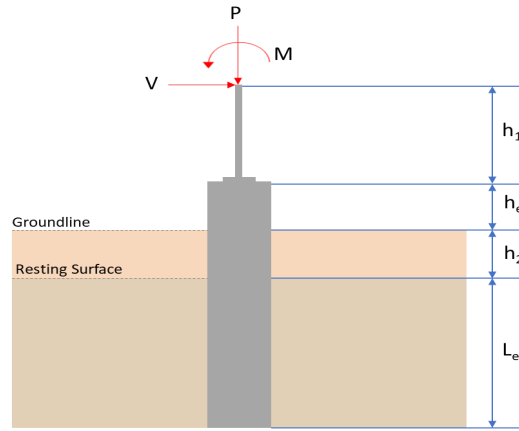
SkyCiv Foundation Design

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

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.548	9.745
V_x (kip)	-1.053	-1.750
V_z (kip)	-0.006	-0.010
M_x (kipft)	-0.027	-0.043
M_z (kipft)	18.534	32.228

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.053 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16768 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.534 \text{ kipft}) + ((-1.053 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9513 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6396 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.006 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00095541 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.027 \text{ kipft}) + ((-0.006 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0042994 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.67329 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6396 \text{ ft}), (0.67329 \text{ ft})]$$

$$L_{e,req} = 5.64 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.64 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.94$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.548 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40925 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40925 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20462$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9513 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9513 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.9513 \text{ kipft/ft})) + (4 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0926 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9513 \text{ kipft/ft})) + (3 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.9513 \text{ kipft/ft})) + (2 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23511 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9513 \text{ kipft/ft})) + ((-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.81608 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0926 \text{ ft})}{2}$$

$$p_a = 0.30694 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23511 \text{ kip/ft}^2)}{(0.30694 \text{ kip/ft}^2)}$$

$$Ratio = 0.76597$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.81608 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.90676$$

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

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = -0.00095541 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0042994 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0042994 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0042994 \text{ kipft/ft})) + (4 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2353 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0042994 \text{ kipft/ft})) + (3 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.0042994 \text{ kipft/ft})) + (2 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 2.7525 \times 10^{-18} \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0042994 \text{ kipft/ft})) + ((-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.00047771 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2353 \text{ ft})}{2}$$

$$p_a = 0.31765 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(2.7525 \times 10^{-18} \text{ kip/ft}^2)}{(0.31765 \text{ kip/ft}^2)}$$

$$Ratio = 8.6651 \times 10^{-18}$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

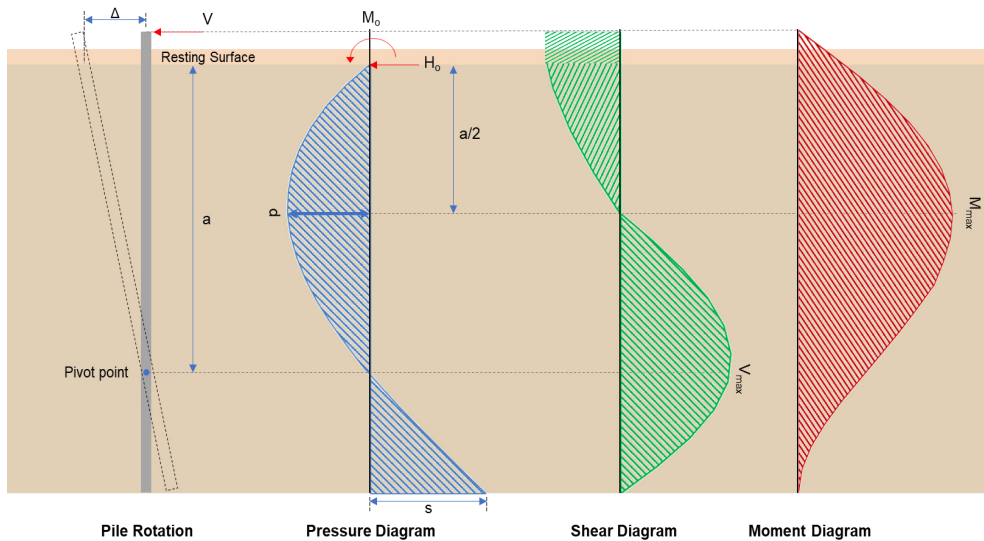
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00047771 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.00053079$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.75 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.228 \text{ kipft}) + ((-1.75 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1318 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1318 \text{ kipft/ft})}{(-0.27866 \text{ kip/ft})}$$

$$E = 18.416 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1318 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.1318 \text{ kipft/ft})) + (4 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.27866 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.1318 \text{ kipft/ft})) + (4 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0892 \text{ ft}$$

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

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

$$V_{max} = ((-0.27866 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0892 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0892 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7951 \text{ kip}$$

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

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

$$M_{max} = ((-0.27866 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(18.416 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.774 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.01 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0015924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.043 \text{ kipft}) + ((-0.01 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0068471 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0068471 \text{ kipft/ft})}{(-0.0015924 \text{ kip/ft})}$$

$$E = 4.3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0068471 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.0015924 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0068471 \text{ kipft/ft})) + (4 \times (-0.0015924 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.241 \text{ ft}$$

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

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

$$V_{max} = ((-0.0015924 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.241 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.241 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.012299 \text{ kip}$$

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

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

$$M_{max} = ((-0.0015924 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(4.3 \text{ ft})}{(6 \text{ ft})} + \frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.033472 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.745 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.272 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.272 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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. 10: 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;">$Ratio = \frac{(9.745 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0036427$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.745 \text{ kip} \rightarrow 9745 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9745 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.78 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.78 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.78 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.78 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.94 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.7951 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.7951 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.06125$$

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

Considering z-direction:

$V_{max} = 0.012299 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.012299 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.00011086$$

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:

$M_{max} = 19.774 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.774 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.079223$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$M_{max} = 0.033472 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.033472 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0001341$$

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

REFERENCES	CALCULATIONS	RESULTS
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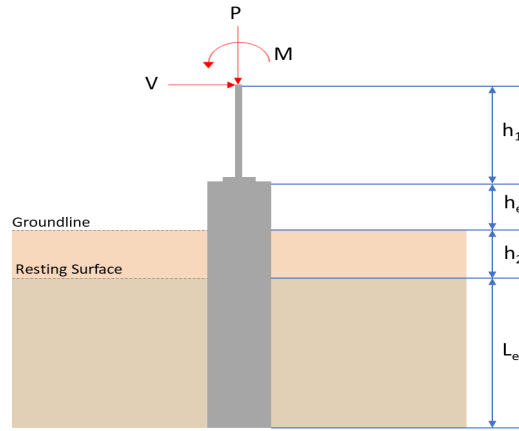
SkyCiv Foundation Design

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

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.469	9.630
V_x (kip)	-1.057	-1.762
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	18.824	32.753

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.057 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16831 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.824 \text{ kipft}) + ((-1.057 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9975 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6724 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6724 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.672 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.672 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.94533$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.469 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20216$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16831 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9975 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9975 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.16831 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.9975 \text{ kipft/ft})) + (4 \times (-0.16831 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0917 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9975 \text{ kipft/ft})) + (3 \times (-0.16831 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.9975 \text{ kipft/ft})) + (2 \times (-0.16831 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23988 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9975 \text{ kipft/ft})) + ((-0.16831 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.83084 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0917 \text{ ft})}{2}$$

$$p_a = 0.30688 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23988 \text{ kip/ft}^2)}{(0.30688 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.78169$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

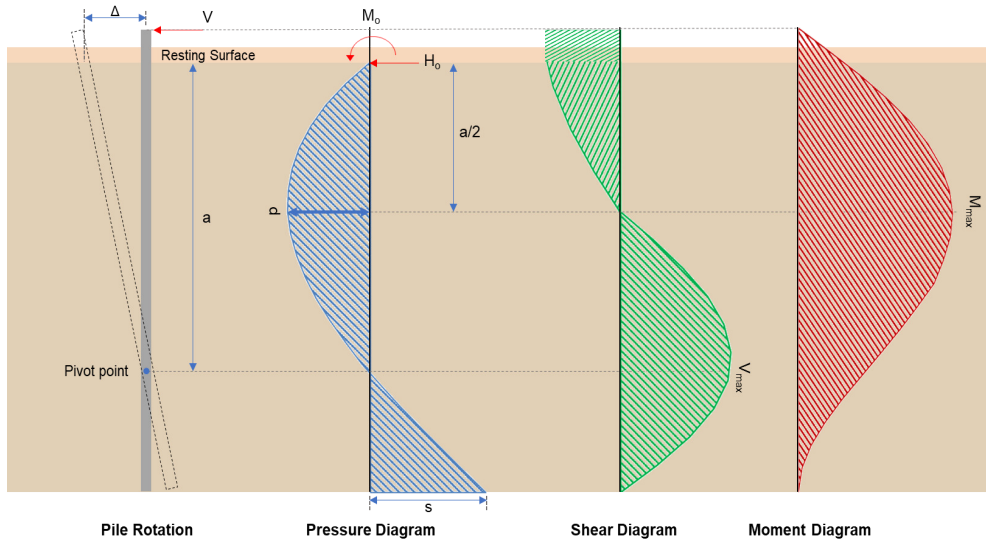
$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.83084 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.780**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.762 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.28057 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.753 \text{ kipft}) + ((-1.762 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.2154 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.2154 \text{ kipft/ft})}{(-0.28057 \text{ kip/ft})}$$

$$E = 18.589 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.2154 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.28057 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.2154 \text{ kipft/ft})) + (4 \times (-0.28057 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0885 \text{ ft}$$

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

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

$$V_{max} = ((-0.28057 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.589 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0885 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.589 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0885 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$v_{max} = 0.099 \text{ kip}$$

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

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

$$M_{max} = ((-0.28057 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(18.589 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0885 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.589 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0885 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (18.589 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0885 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.081 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.63 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.276 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.276 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

<p>25.7.2.2 25.7.2.1</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: Since longitudinal reinforcement is \leq No. 10@: Use #3(0.375 in) s_{ties} - Maximum spacing of 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>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.63 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0035998$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> <p style="text-align: center;">$V_{c,max} = 296.21 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.63 \text{ kip} \rightarrow 9630 \text{ lbf}$,</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> <p style="text-align: center;">$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$</p>	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9630 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.77 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.77 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.77 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.77 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.93 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.899 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.899 \text{ kip})}{(110.93 \text{ kip})}$$

$$\text{Ratio} = 0.062192$$

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

Flexural Strength (ACI 318-19, LFRD)

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:

$M_{max} = 20.081 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(20.081 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.080455$$

Status: **PASS**
Ratio: **0.080**

REFERENCES	CALCULATIONS	RESULTS
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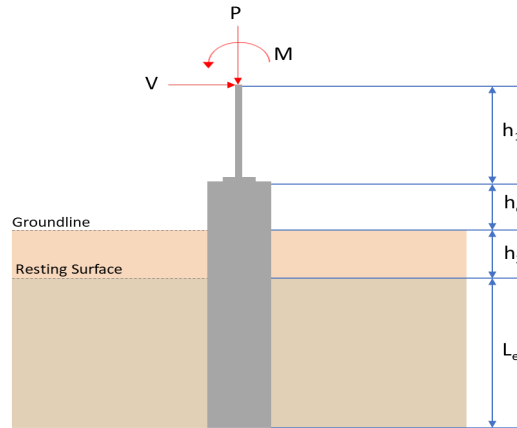
SkyCiv Foundation Design

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

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.548	9.745
V_x (kip)	-1.053	-1.750
V_z (kip)	0.006	0.010
M_x (kipft)	0.027	0.043
M_z (kipft)	18.534	32.228

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.053 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16768 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.534 \text{ kipft}) + ((-1.053 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9513 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6396 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.006 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00095541 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.027 \text{ kipft}) + ((0.006 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0042994 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.72807 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6396 \text{ ft}), (0.72807 \text{ ft})]$$

$$L_{e,req} = 5.64 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(5.64 \text{ ft})}{(6 \text{ ft})}$$

$$Ratio = 0.94$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.548 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40925 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40925 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20462$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9513 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9513 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.9513 \text{ kipft/ft})) + (4 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0926 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9513 \text{ kipft/ft})) + (3 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.9513 \text{ kipft/ft})) + (2 \times (-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23511 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9513 \text{ kipft/ft})) + ((-0.16768 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.81608 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0926 \text{ ft})}{2}$$

$$p_a = 0.30694 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23511 \text{ kip/ft}^2)}{(0.30694 \text{ kip/ft}^2)}$$

$$Ratio = 0.76597$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.81608 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.90676$$

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

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = 0.00095541 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0042994 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0042994 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0042994 \text{ kipft/ft})) + (4 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2353 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0042994 \text{ kipft/ft})) + (3 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 [(3 \times (0.0042994 \text{ kipft/ft})) + (2 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.0010116 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0042994 \text{ kipft/ft})) + ((0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.0023885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2353 \text{ ft})}{2}$$

$$p_a = 0.31765 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0010116 \text{ kip/ft}^2)}{(0.31765 \text{ kip/ft}^2)}$$

$$Ratio = 0.0031847$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

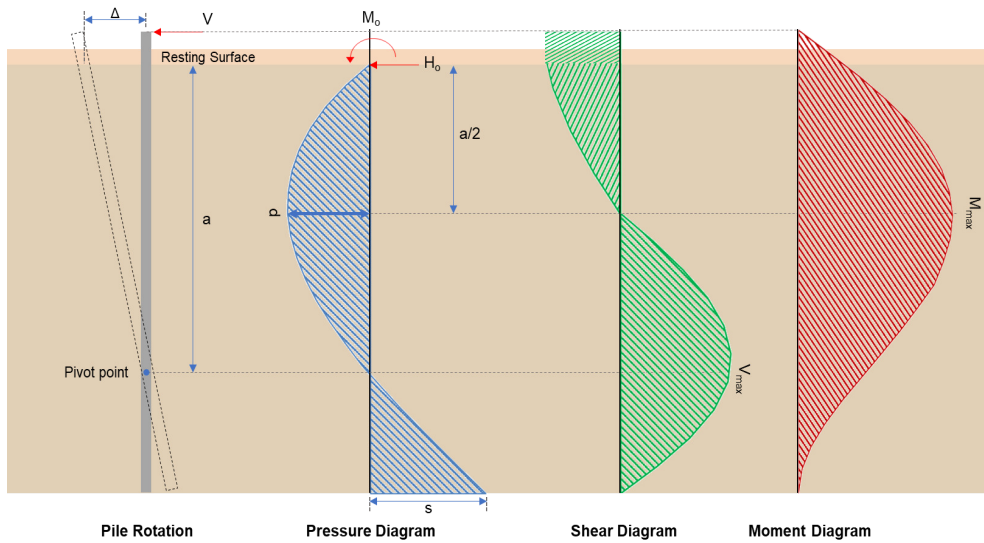
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0023885 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.0026539$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.75 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.228 \text{ kipft}) + ((-1.75 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1318 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1318 \text{ kipft/ft})}{(-0.27866 \text{ kip/ft})}$$

$$E = 18.416 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1318 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.1318 \text{ kipft/ft})) + (4 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.27866 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.1318 \text{ kipft/ft})) + (4 \times (-0.27866 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0892 \text{ ft}$$

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

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

$$V_{max} = ((-0.27866 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0892 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0892 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7951 \text{ kip}$$

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

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

$$M_{max} = ((-0.27866 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(18.416 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.416 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0892 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.774 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.01 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0015924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.043 \text{ kipft}) + ((0.01 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0068471 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0068471 \text{ kipft/ft})}{(0.0015924 \text{ kip/ft})}$$

$$E = 4.3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0068471 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.0015924 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0068471 \text{ kipft/ft})) + (4 \times (0.0015924 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.241 \text{ ft}$$

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

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

$$V_{max} = ((0.0015924 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.241 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.241 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.012299 \text{ kip}$$

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

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

$$M_{max} = ((0.0015924 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(4.3 \text{ ft})}{(6 \text{ ft})} + \frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.3 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.241 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.033472 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.745 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.272 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.272 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.745 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0036427$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.745 \text{ kip} \rightarrow 9745 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9745 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.78 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.78 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.78 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.78 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.94 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.7951 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.7951 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.06125$$

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

Considering z-direction:

$V_{max} = 0.012299 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.012299 \text{ kip})}{(110.94 \text{ kip})}$$

$$Ratio = 0.00011086$$

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:

$M_{max} = 19.774 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.774 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.079223$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$M_{max} = 0.033472 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.033472 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0001341$$

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