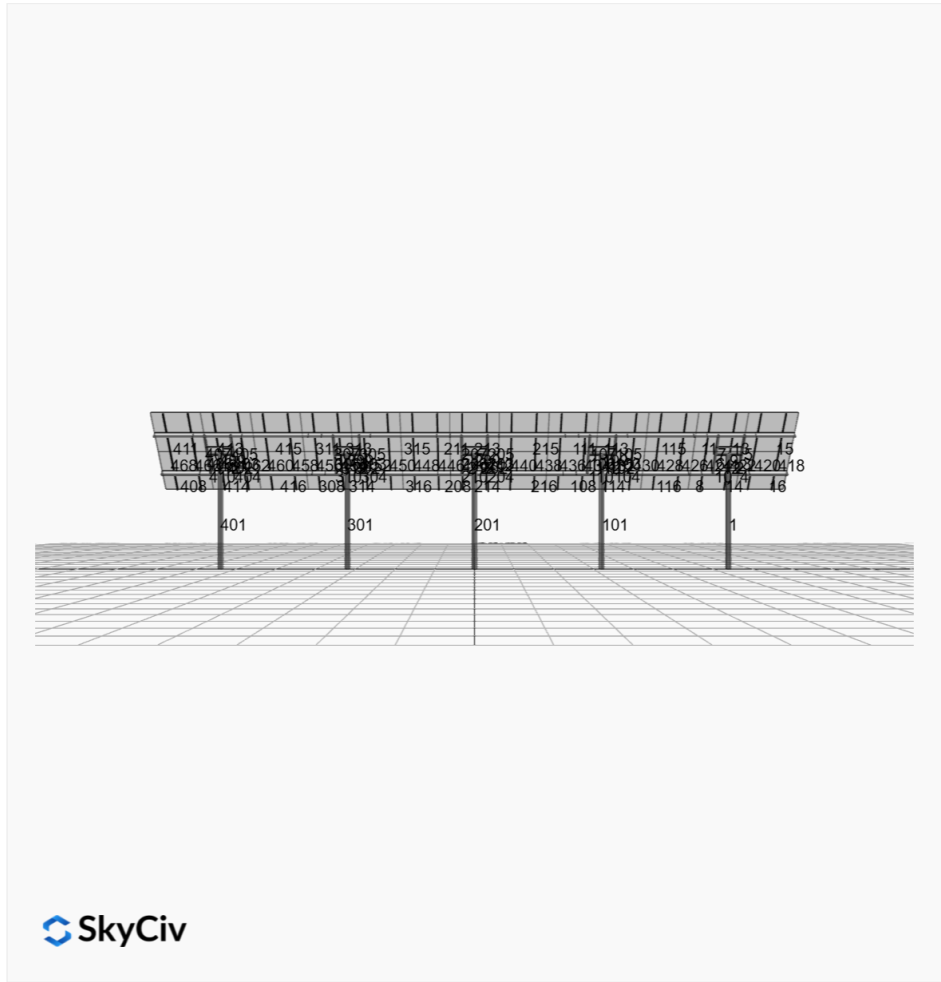


Project Name: MTSOLAR_5ADE89IB64157 - Mill Meter
Date: Fri Feb 28 2025
Location: Fort Collins, CO, USA
Number of Modules: 52
Unique ID: 5P-19.75-8TOP-XD-72-L-4Hx13W-64G9
Number of Poles: 5
Date Sold:
Dealer: _____



Array Dimensions N/S	15.17 ft
Array Dimensions E/W	98.58 ft
Winter Tilt Angle	50
Front Edge Clearance	12 ft

MT Solar Bill of Materials (5P-19.75-8TOP-XD-72-L-4Hx13W-64G9)

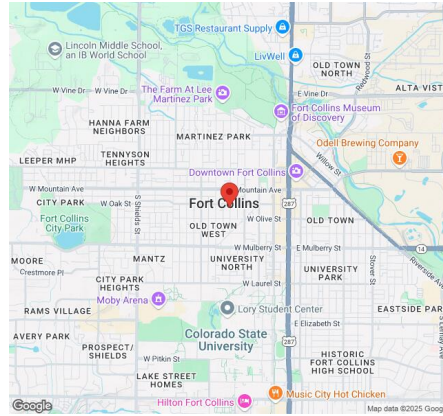
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-XD	H-Frame Assembly-XD	5
MTS-XD-Wing-72	72IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	8
MTS-XD-Splice-57	57IN XD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	13

Rail Bill of Materials

Part	Qty
------	-----

Part	Qty
Rails (180in)	26
Rail Attachment	52
Module Mid Clamp	78
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: Fort Collins, CO, USA

Array Specification

Duty Classification:	XD
Module Width:	45.00 in
Module Length:	90.00in
Number of Rows:	4
Number of Columns:	13
Total Number of Modules:	52
Winter Tilt Angle:	50
Front Edge Clearance:	12
Total Array Height at Tilt:	23.62 ft
Total Frame Length:	98.50 ft
Frame Weight:	7382 lbs
Array Dimensions N/S:	15.17 ft
Array Dimensions E/W:	98.58 ft
Rail Length:	182.00 in
Rail Spacing:	3.79 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	17.81 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: 5.00 ft Pile 2: 4.75 ft Pile 3: 4.75 ft Pile 4: 4.75 ft Pile 5: 5.00 ft
Foundation Volume:	14.370 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Fort Collins, CO, USA
Wind Speed:	45 mph

Snow Load:

90 psf

Design Disclaimer

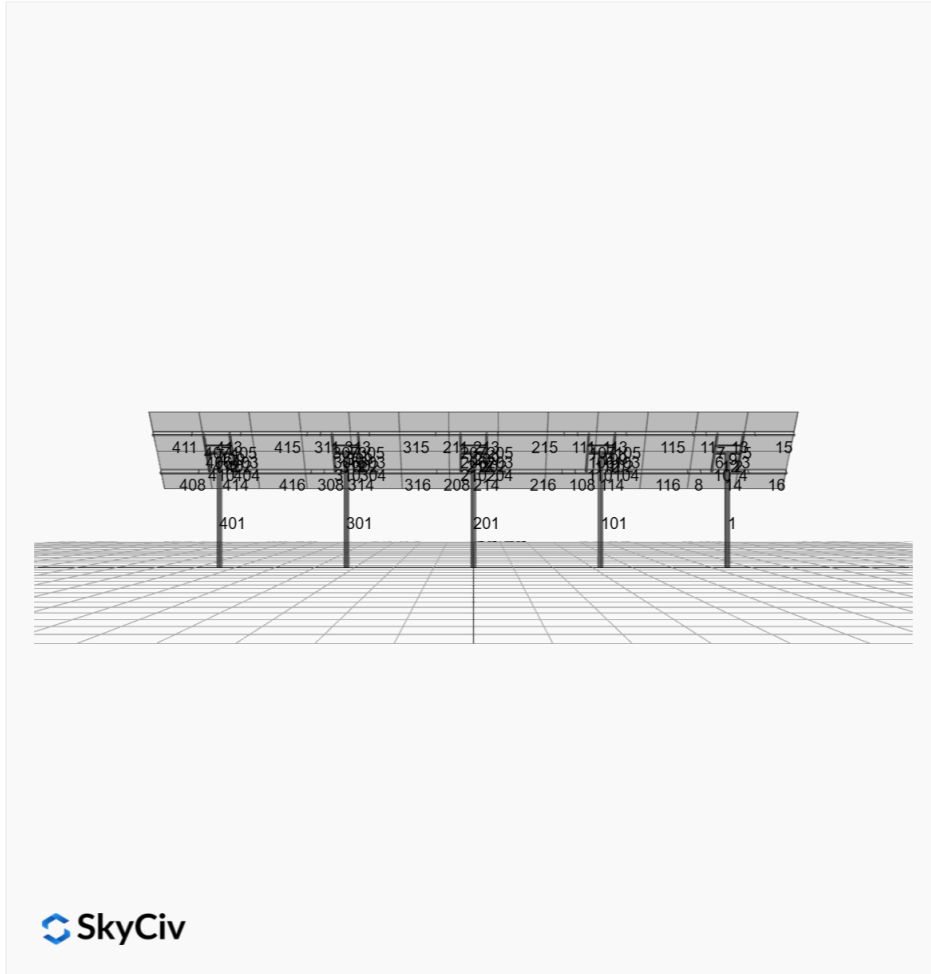
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

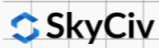
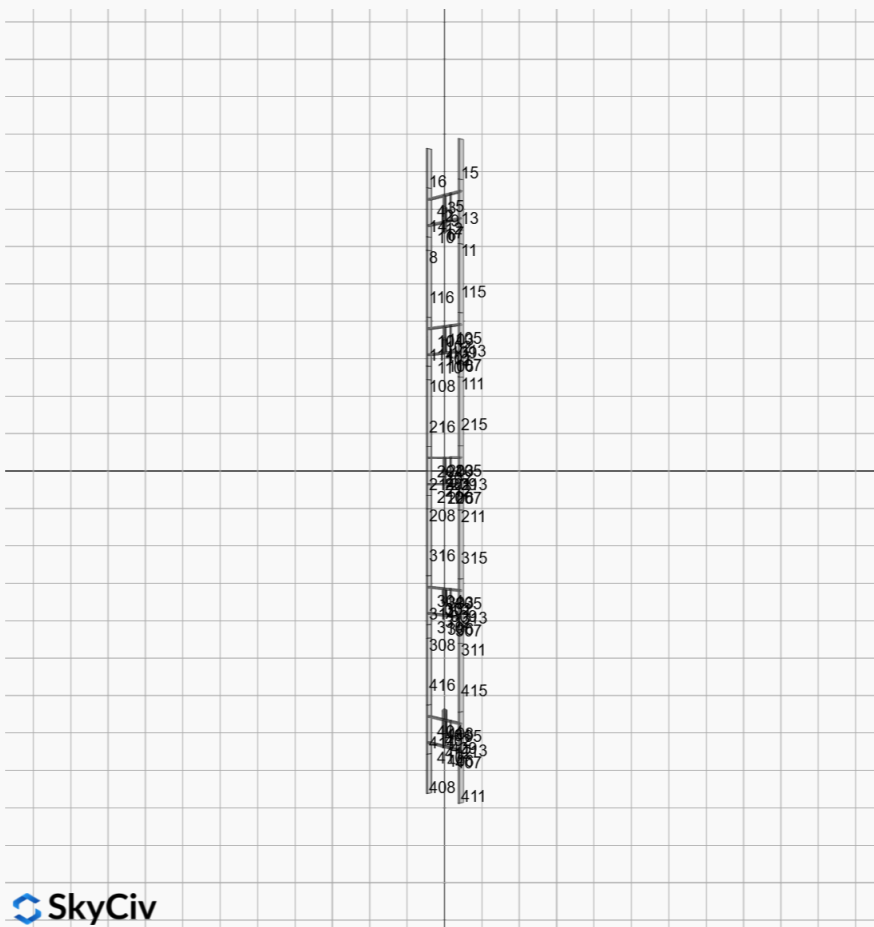
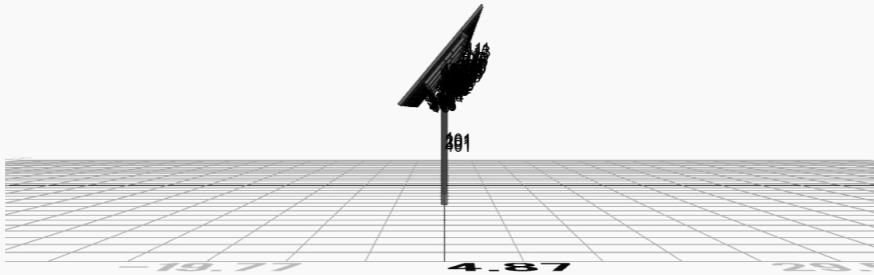
AutoDesigner Input

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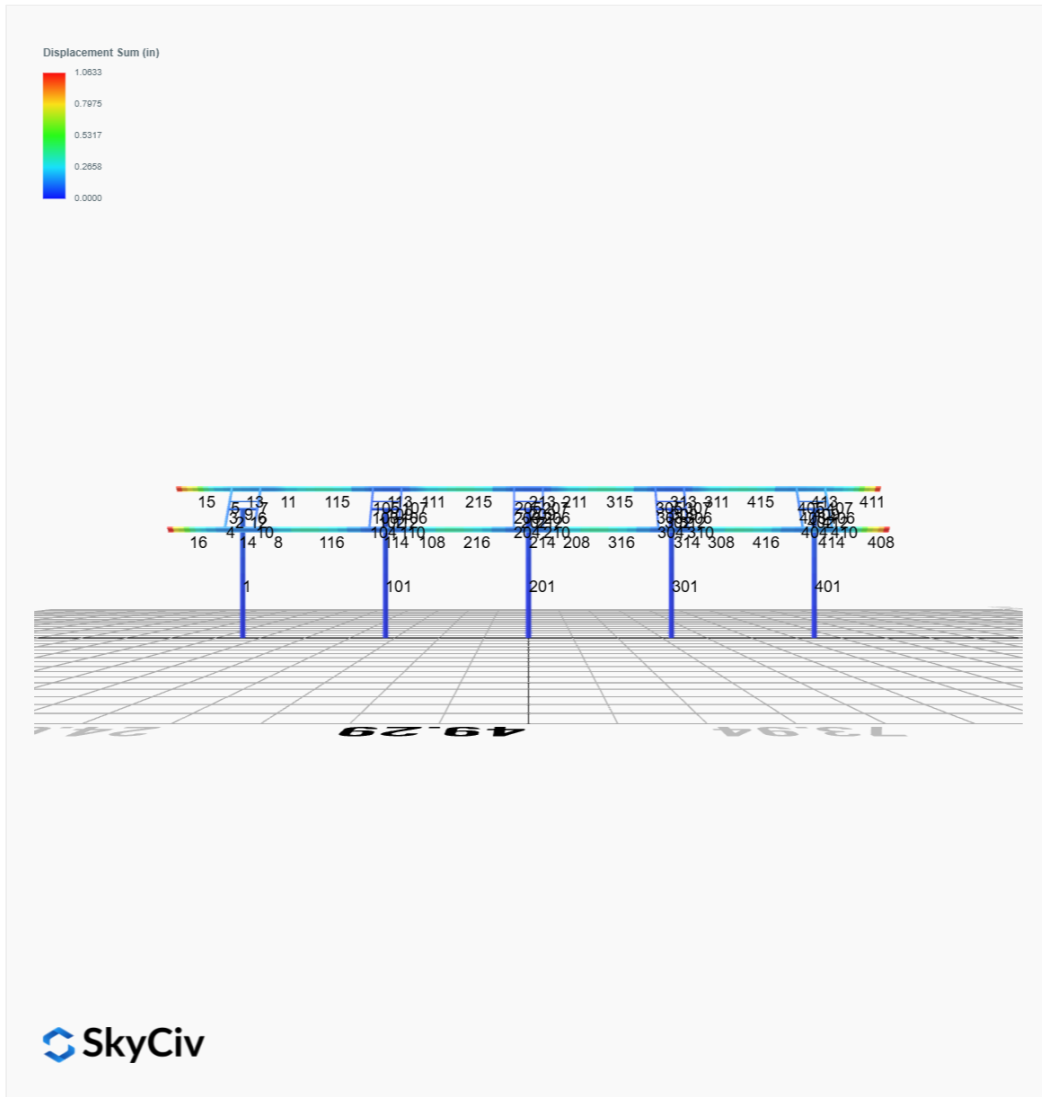
Design Notes:

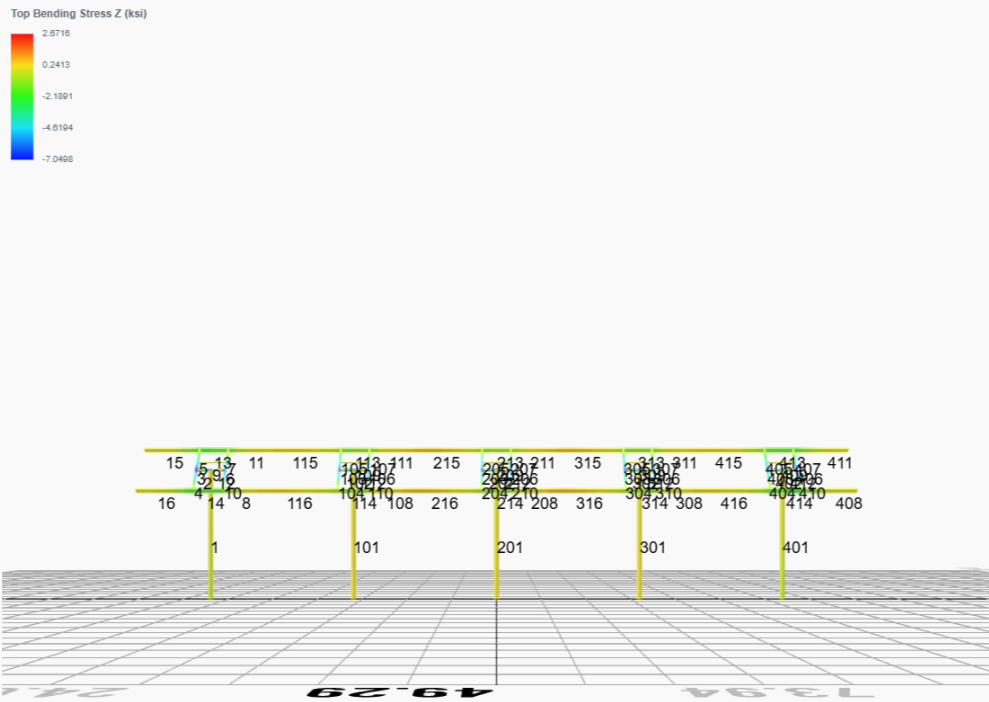
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

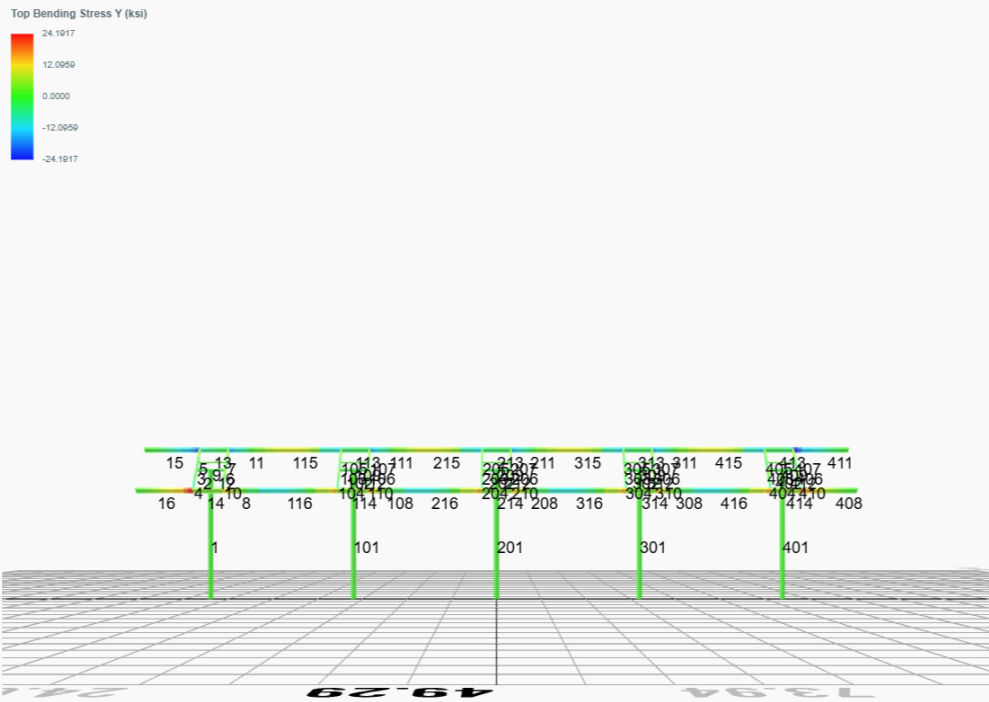


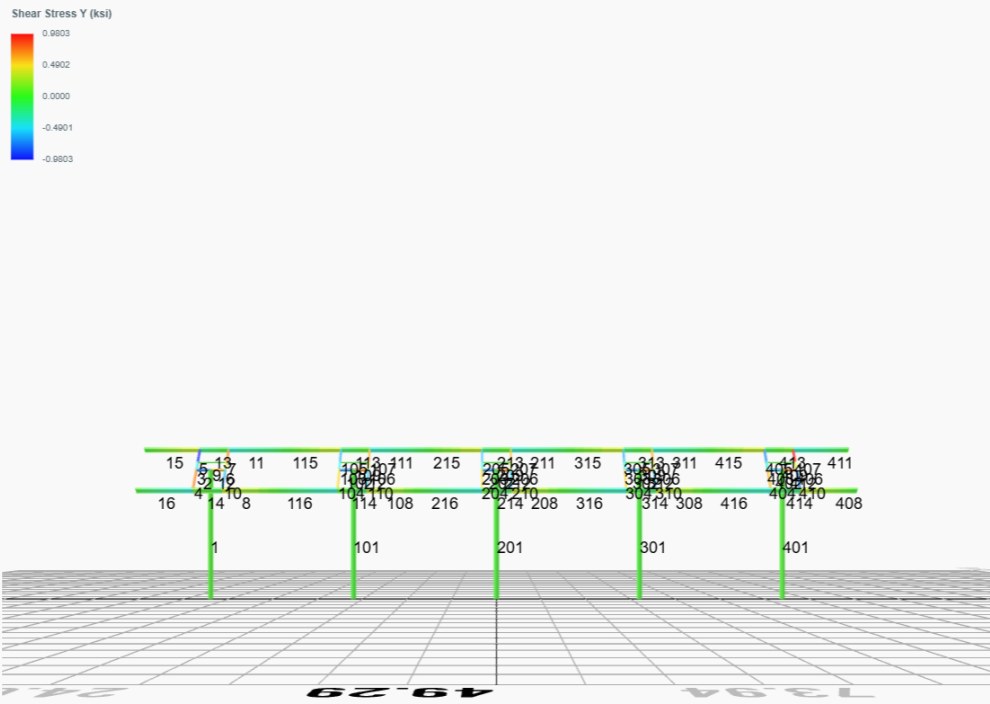


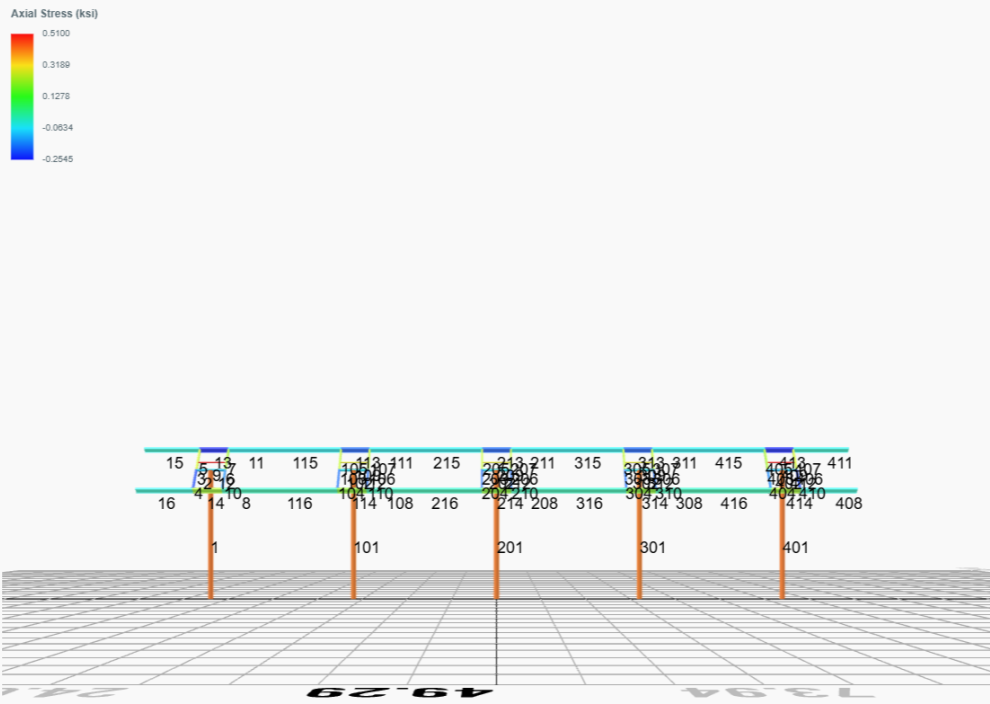
FEM Results (Envelope Worst Case for each member)











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	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 2. D + L	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 3. D + (S or Lr or R)	-0.0477	6.4446	-0.0733	-0.4214	0.2845	0.8280
ULS: 3. D + (S or Lr or R)	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0397	5.4881	-0.0609	-0.3502	0.2365	0.6907
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 5b. D + 0.7E	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0397	5.4881	-0.0609	-0.3502	0.2365	0.6907
ULS: 8. 0.6D + 0.7E	-0.0094	1.5711	-0.0143	-0.0820	0.0554	0.1673
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5452	3.0657	-0.0349	-0.1998	0.1544	9.7476
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5140	2.1711	-0.0128	-0.0737	0.0307	-9.0902
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4369	5.8235	-0.0692	-0.3977	0.2830	7.7922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0397	5.4881	-0.0609	-0.3502	0.2365	0.6907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3575	5.1526	-0.0526	-0.3031	0.1902	-6.3361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0397	5.4881	-0.0609	-0.3502	0.2365	0.6907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4128	2.9539	-0.0321	-0.1840	0.1389	7.3804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3816	2.2830	-0.0155	-0.0894	0.0461	-6.7479
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0156	2.6184	-0.0238	-0.1366	0.0924	0.2789
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5389	2.0183	-0.0254	-0.1452	0.1175	9.6360
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0094	1.5711	-0.0143	-0.0820	0.0554	0.1673
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5203	1.1238	-0.0032	-0.0191	-0.0063	-9.2017
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0094	1.5711	-0.0143	-0.0820	0.0554	0.1673

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	9.6361
Shear X	-0.9167
Shear Z	-0.1181
Moment X	-0.6820
Moment Y (Twist)	0.4753
Moment Z	17.0171

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	6.4446
Shear X	-0.5452
Shear Z	-0.0733
Moment X	-0.4214
Moment Y (Twist)	0.2845
Moment Z	9.7476

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	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 2. D + L	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 3. D + (S or Lr or R)	0.0360	6.3742	-0.0029	-0.0149	0.0123	-0.5134
ULS: 3. D + (S or Lr or R)	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0299	5.4295	-0.0024	-0.0124	0.0102	-0.4240

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 5b. D + 0.7E	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0299	5.4295	-0.0024	-0.0124	0.0102	-0.4240
ULS: 8. 0.6D + 0.7E	0.0071	1.5572	-0.0005	-0.0028	0.0023	-0.0935
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5032	3.0245	-0.0008	-0.0042	0.0056	9.0824
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5267	2.1664	-0.0010	-0.0054	0.0023	-9.2967
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3564	5.7513	-0.0023	-0.0120	0.0115	6.5047
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0299	5.4295	-0.0024	-0.0124	0.0102	-0.4240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4161	5.1077	-0.0025	-0.0129	0.0091	-7.2796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0299	5.4295	-0.0024	-0.0124	0.0102	-0.4240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3745	2.9172	-0.0009	-0.0043	0.0051	6.7728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3980	2.2736	-0.0010	-0.0053	0.0027	-7.0115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0118	2.5954	-0.0009	-0.0047	0.0038	-0.1559
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5080	1.9863	-0.0005	-0.0023	0.0040	9.1447
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0071	1.5572	-0.0005	-0.0028	0.0023	-0.0935
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5220	1.1282	-0.0007	-0.0036	0.0008	-9.2343
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0071	1.5572	-0.0005	-0.0028	0.0023	-0.0935

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	9.5188
Shear X	-0.8845
Shear Z	-0.0049
Moment X	-0.0256
Moment Y (Twist)	0.0242
Moment Z	16.1803

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	6.3742
Shear X	-0.5267
Shear Z	-0.0029
Moment X	-0.0149
Moment Y (Twist)	0.0123
Moment Z	9.2967

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	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 2. D + L	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 3. D + (S or Lr or R)	0.0235	6.4015	-0.0000	0.0000	-0.0000	-0.4373
ULS: 3. D + (S or Lr or R)	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0196	5.4522	-0.0000	0.0000	-0.0000	-0.3605
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 5b. D + 0.7E	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0196	5.4522	-0.0000	0.0000	-0.0000	-0.3605
ULS: 8. 0.6D + 0.7E	0.0046	1.5626	-0.0000	0.0000	0.0000	-0.0781
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5102	3.0392	-0.0000	0.0000	0.0000	9.1315
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5256	2.1694	-0.0000	0.0000	0.0000	-9.2955
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302

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	-0.3688	5.7784	-0.0000	0.0000	-0.0000	6.5858
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0196	5.4522	-0.0000	0.0000	-0.0000	-0.3605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4080	5.1260	-0.0000	0.0000	-0.0000	-7.2345
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0196	5.4522	-0.0000	0.0000	-0.0000	-0.3605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3807	2.9305	-0.0000	0.0000	0.0000	6.8161
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3961	2.2782	-0.0000	0.0000	0.0000	-7.0042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0077	2.6043	-0.0000	0.0000	0.0000	-0.1302
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5132	1.9975	-0.0000	0.0000	0.0000	9.1836
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0046	1.5626	-0.0000	0.0000	0.0000	-0.0781
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5225	1.1277	-0.0000	0.0000	0.0000	-9.2434
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0046	1.5626	-0.0000	0.0000	0.0000	-0.0781

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	9.5629
Shear X	-0.8802
Shear Z	-0.0000
Moment X	0.0002
Moment Y (Twist)	0.0001
Moment Z	16.1643

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	6.4015
Shear X	-0.5256
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	9.2955

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	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 2. D + L	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 3. D + (S or Lr or R)	0.0360	6.3742	0.0029	0.0150	-0.0124	-0.5134
ULS: 3. D + (S or Lr or R)	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0299	5.4295	0.0024	0.0124	-0.0102	-0.4240
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 5b. D + 0.7E	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0299	5.4295	0.0024	0.0124	-0.0102	-0.4240
ULS: 8. 0.6D + 0.7E	0.0071	1.5572	0.0005	0.0028	-0.0023	-0.0935
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5032	3.0245	0.0008	0.0042	-0.0056	9.0824
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5267	2.1664	0.0010	0.0055	-0.0023	-9.2967
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3564	5.7513	0.0023	0.0120	-0.0115	6.5047
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0299	5.4295	0.0024	0.0124	-0.0102	-0.4240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4161	5.1077	0.0025	0.0130	-0.0091	-7.2796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0299	5.4295	0.0024	0.0124	-0.0102	-0.4240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3745	2.9172	0.0009	0.0043	-0.0051	6.7728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3980	2.2736	0.0010	0.0053	-0.0027	-7.0115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0118	2.5954	0.0009	0.0047	-0.0038	-0.1559

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5080	1.9863	0.0005	0.0023	-0.0040	9.1447
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0071	1.5572	0.0005	0.0028	-0.0023	-0.0935
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5220	1.1282	0.0007	0.0036	-0.0008	-9.2343
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0071	1.5572	0.0005	0.0028	-0.0023	-0.0935

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	9.5188
Shear X	-0.8845
Shear Z	0.0049
Moment X	0.0259
Moment Y (Twist)	0.0243
Moment Z	16.1805

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	6.3742
Shear X	-0.5267
Shear Z	0.0029
Moment X	0.0150
Moment Y (Twist)	0.0124
Moment Z	9.2967

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	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 2. D + L	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 3. D + (S or Lr or R)	-0.0477	6.4446	0.0733	0.4215	-0.2846	0.8280
ULS: 3. D + (S or Lr or R)	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0397	5.4881	0.0609	0.3503	-0.2365	0.6907
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 5b. D + 0.7E	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0397	5.4881	0.0609	0.3503	-0.2365	0.6907
ULS: 8. 0.6D + 0.7E	-0.0094	1.5711	0.0143	0.0820	-0.0554	0.1673
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5452	3.0657	0.0349	0.1999	-0.1544	9.7476
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5140	2.1711	0.0128	0.0737	-0.0307	-9.0902
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4369	5.8235	0.0692	0.3977	-0.2830	7.7922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0397	5.4881	0.0609	0.3503	-0.2365	0.6907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3575	5.1526	0.0526	0.3031	-0.1902	-6.3361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0397	5.4881	0.0609	0.3503	-0.2365	0.6907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4128	2.9539	0.0321	0.1840	-0.1389	7.3804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3816	2.2830	0.0155	0.0895	-0.0461	-6.7479
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0156	2.6184	0.0238	0.1366	-0.0924	0.2789
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5389	2.0183	0.0254	0.1452	-0.1175	9.6360
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0094	1.5711	0.0143	0.0820	-0.0554	0.1673
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5203	1.1238	0.0032	0.0191	0.0063	-9.2017
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0094	1.5711	0.0143	0.0820	-0.0554	0.1673

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	9.6361
Shear X	-0.9167
Shear Z	0.1181
Moment X	0.6824
Moment Y (Twist)	0.4754
Moment Z	17.0174

Result	Value (kip, kip-ft)
Axial	6.4446
Shear X	-0.5452
Shear Z	0.0733
Moment X	0.4215
Moment Y (Twist)	0.2846
Moment Z	9.7476

Project Details

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

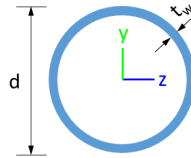


Design Input Information

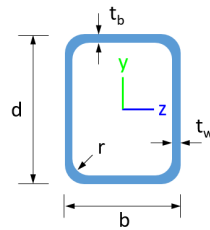
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

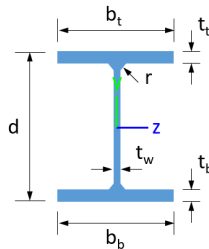
Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	246.00	27.16	27.16	75.30	75.30
113	159.30	97.43	32.15	6.46	56.26	44.91
114	159.30	97.43	32.32	6.46	56.26	44.91
115	159.30	75.13	23.47	6.46	56.26	44.91
116	159.30	75.13	22.77	6.46	56.26	44.91
201	377.97	81.31	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.63	6.46	56.26	44.91
214	159.30	97.43	31.66	6.46	56.26	44.91
215	159.30	75.13	21.56	6.46	56.26	44.91
216	159.30	75.13	21.94	6.46	56.26	44.91
301	377.97	81.31	83.29	83.29	113.39	113.39
302	251.01	246.00	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	140.46	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	140.46	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	32.21	6.46	56.26	44.91
314	159.30	97.43	32.25	6.46	56.26	44.91
315	159.30	75.13	22.21	6.46	56.26	44.91
316	159.30	75.13	22.10	6.46	56.26	44.91
401	377.97	81.31	83.29	83.29	113.39	113.39
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95
407	151.65	145.15	20.17	14.14	54.12	28.95

407	151.05	149.10	20.17	14.14	54.12	28.95
408	159.30	21.54	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	21.54	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	97.43	32.68	6.46	56.26	44.91
414	159.30	97.43	32.89	6.46	56.26	44.91
415	159.30	75.13	24.75	6.46	56.26	44.91
416	159.30	75.13	26.64	6.46	56.26	44.91

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.119	0.204	0.017	0.008	0.001	0.245	#13	0.764	Not Required	Pass
2	0.005	0.264	0.072	0.065	0.006	0.339	#21	0.036	Not Required	Pass
3	0.012	0.306	0.065	0.031	0.005	0.377	#21	0.046	Not Required	Pass
4	0.012	0.303	0.255	0.030	0.052	0.453	#21	0.082	Not Required	Pass
5	0.012	0.189	0.267	0.030	0.068	0.273	#23	0.076	Not Required	Pass
6	0.010	0.260	0.060	0.026	0.007	0.304	#21	0.046	Not Required	Pass
7	0.010	0.161	0.204	0.026	0.056	0.227	#21	0.076	Not Required	Pass
8	0.002	0.039	0.202	0.018	0.022	0.242	#21	0.102	Not Required	Pass
9	0.027	0.013	0.045	0.001	0.001	0.071	#21	0.206	Not Required	Pass
10	0.010	0.264	0.201	0.026	0.044	0.404	#21	0.082	Not Required	Pass
11	0.002	0.036	0.204	0.017	0.022	0.241	#21	0.068	Not Required	Pass
12	0.006	0.212	0.055	0.055	0.006	0.270	#21	0.036	Not Required	Pass
13	0.012	0.139	0.748	0.022	0.028	0.884	#21	0.204	Not Required	Pass
14	0.015	0.144	0.748	0.022	0.028	0.884	#21	0.204	Not Required	Pass
15	0.000	0.057	0.448	0.016	0.021	0.506	#21	Not Required	Not Required	Pass
16	0.000	0.057	0.448	0.016	0.021	0.506	#21	Not Required	Not Required	Pass
101	0.117	0.194	0.001	0.008	0.000	0.223	#13	0.764	Not Required	Pass
102	0.003	0.236	0.055	0.059	0.006	0.293	#21	0.036	Not Required	Pass
103	0.011	0.280	0.066	0.028	0.010	0.351	#21	0.046	Not Required	Pass
104	0.011	0.271	0.174	0.027	0.038	0.392	#21	0.082	Not Required	Pass
105	0.011	0.173	0.178	0.028	0.046	0.219	#21	0.076	Not Required	Pass
106	0.011	0.276	0.069	0.028	0.011	0.350	#21	0.046	Not Required	Pass
107	0.011	0.171	0.183	0.027	0.047	0.217	#21	0.076	Not Required	Pass
108	0.002	0.020	0.192	0.016	0.022	0.206	#21	0.102	Not Required	Pass
109	0.016	0.013	0.030	0.001	0.000	0.050	#21	0.206	Not Required	Pass
110	0.011	0.272	0.179	0.027	0.039	0.397	#21	0.082	Not Required	Pass
111	0.002	0.017	0.193	0.017	0.022	0.211	#21	0.068	Not Required	Pass
112	0.003	0.234	0.055	0.059	0.006	0.290	#21	0.055	Not Required	Pass
113	0.011	0.098	0.498	0.021	0.028	0.582	#21	0.204	Not Required	Pass
114	0.014	0.091	0.498	0.021	0.028	0.575	#21	0.306	Not Required	Pass
115	0.002	0.058	0.271	0.016	0.022	0.330	#21	0.338	Not Required	Pass
116	0.003	0.063	0.277	0.015	0.022	0.340	#21	0.507	Not Required	Pass
201	0.118	0.194	0.000	0.008	0.000	0.224	#13	0.764	Not Required	Pass
202	0.003	0.236	0.055	0.059	0.006	0.293	#21	0.036	Not Required	Pass
203	0.011	0.277	0.067	0.028	0.010	0.349	#21	0.046	Not Required	Pass
204	0.011	0.276	0.178	0.028	0.039	0.401	#21	0.082	Not Required	Pass
205	0.011	0.171	0.183	0.027	0.047	0.218	#21	0.076	Not Required	Pass

206	0.011	0.277	0.067	0.028	0.010	0.349	#21	0.046	Not Required	Pass
207	0.011	0.171	0.183	0.027	0.047	0.218	#21	0.076	Not Required	Pass
208	0.002	0.022	0.192	0.016	0.022	0.203	#21	0.102	Not Required	Pass
209	0.016	0.011	0.029	0.001	0.000	0.048	#21	0.206	Not Required	Pass
210	0.011	0.276	0.178	0.028	0.039	0.401	#21	0.082	Not Required	Pass
211	0.002	0.020	0.192	0.016	0.022	0.206	#21	0.068	Not Required	Pass
212	0.003	0.236	0.055	0.059	0.006	0.293	#21	0.036	Not Required	Pass
213	0.011	0.085	0.497	0.021	0.028	0.574	#21	0.204	Not Required	Pass
214	0.014	0.085	0.497	0.021	0.028	0.572	#21	0.306	Not Required	Pass
215	0.002	0.094	0.271	0.016	0.022	0.366	#21	0.338	Not Required	Pass
216	0.003	0.101	0.277	0.016	0.022	0.378	#21	0.507	Not Required	Pass
301	0.117	0.194	0.001	0.008	0.000	0.223	#13	0.764	Not Required	Pass
302	0.003	0.234	0.055	0.059	0.006	0.290	#21	0.055	Not Required	Pass
303	0.011	0.276	0.068	0.028	0.011	0.350	#21	0.046	Not Required	Pass
304	0.011	0.272	0.179	0.027	0.039	0.397	#21	0.082	Not Required	Pass
305	0.011	0.171	0.183	0.027	0.047	0.217	#21	0.076	Not Required	Pass
306	0.011	0.280	0.066	0.028	0.010	0.351	#21	0.046	Not Required	Pass
307	0.011	0.173	0.178	0.028	0.046	0.219	#21	0.076	Not Required	Pass
308	0.002	0.019	0.183	0.015	0.022	0.201	#21	0.102	Not Required	Pass
309	0.016	0.013	0.030	0.001	0.000	0.050	#21	0.206	Not Required	Pass
310	0.011	0.271	0.174	0.027	0.038	0.392	#21	0.082	Not Required	Pass
311	0.002	0.025	0.183	0.016	0.022	0.207	#21	0.068	Not Required	Pass
312	0.003	0.236	0.056	0.059	0.006	0.293	#21	0.036	Not Required	Pass
313	0.011	0.098	0.498	0.021	0.028	0.582	#21	0.204	Not Required	Pass
314	0.014	0.091	0.498	0.021	0.028	0.575	#21	0.306	Not Required	Pass
315	0.002	0.092	0.271	0.017	0.022	0.365	#21	0.338	Not Required	Pass
316	0.003	0.100	0.277	0.016	0.022	0.378	#21	0.507	Not Required	Pass
401	0.119	0.204	0.017	0.008	0.001	0.245	#13	0.764	Not Required	Pass
402	0.006	0.212	0.055	0.055	0.006	0.270	#21	0.036	Not Required	Pass
403	0.010	0.260	0.060	0.026	0.007	0.304	#21	0.046	Not Required	Pass
404	0.010	0.264	0.201	0.026	0.044	0.404	#21	0.082	Not Required	Pass
405	0.010	0.161	0.204	0.026	0.056	0.227	#21	0.076	Not Required	Pass
406	0.012	0.306	0.065	0.031	0.005	0.377	#21	0.046	Not Required	Pass
407	0.012	0.189	0.267	0.030	0.068	0.273	#23	0.076	Not Required	Pass
408	0.000	0.057	0.448	0.016	0.021	0.506	#21	Not Required	Not Required	Pass
409	0.027	0.013	0.045	0.001	0.001	0.071	#21	0.206	Not Required	Pass
410	0.012	0.303	0.255	0.030	0.052	0.453	#21	0.082	Not Required	Pass
411	0.000	0.057	0.448	0.016	0.021	0.506	#21	Not Required	Not Required	Pass
412	0.005	0.264	0.072	0.065	0.006	0.339	#21	0.036	Not Required	Pass
413	0.012	0.139	0.748	0.022	0.028	0.884	#21	0.204	Not Required	Pass
414	0.015	0.144	0.748	0.022	0.028	0.884	#21	0.306	Not Required	Pass
415	0.002	0.058	0.272	0.017	0.022	0.320	#21	0.338	Not Required	Pass
416	0.003	0.060	0.278	0.018	0.022	0.327	#21	0.507	Not Required	Pass

Definitions

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

A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/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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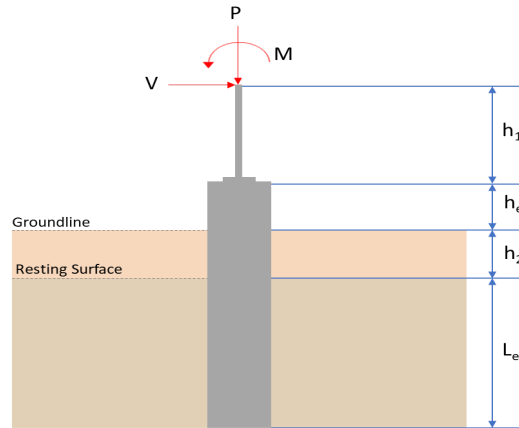
SkyCiv Foundation Design

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

$L = 4.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.374	9.519
V_x (kip)	-0.527	-0.885
V_z (kip)	-0.003	-0.005
M_x (kipft)	-0.015	-0.026
M_z (kipft)	9.297	16.180

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,

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

$$H_o = \frac{(-0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.083917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.297 \text{ kipft}) + ((-0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4804 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.5697 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.003 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.015 \text{ kipft}) + ((-0.003 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0023885 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.55944 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.5697 \text{ ft}), (0.55944 \text{ ft})]$$

$$L_{e,req} = 4.57 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.57 \text{ ft})}{(4.75 \text{ ft})}$$

$$\text{Ratio} = 0.96211$$

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

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

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

$$q = \frac{(6.374 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39838 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39838 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1992$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.083917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4804 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4804 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.4804 \text{ kipft/ft})) + (4 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2269 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4804 \text{ kipft/ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.4804 \text{ kipft/ft})) + (2 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20373 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4804 \text{ kipft/ft})) + ((-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.68137 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2269 \text{ ft})}{2}$$

$$p_a = 0.24202 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20373 \text{ kip/ft}^2)}{(0.24202 \text{ kip/ft}^2)}$$

$$Ratio = 0.84179$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.68137 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.9563$$

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

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

Considering z-direction:

$H_o = -0.00047771 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0023885 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0023885 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.0023885 \text{ kipft/ft})) + (4 \times (-0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3202 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0023885 \text{ kipft/ft})) + (3 \times (-0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (0.0023885 \text{ kipft/ft})) + (2 \times (-0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.000095457 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0023885 \text{ kipft/ft})) + ((-0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.00066694 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3202 \text{ ft})}{2}$$

$$p_a = 0.24901 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000095457 \text{ kip/ft}^2)}{(0.24901 \text{ kip/ft}^2)}$$

$$Ratio = 0.00038334$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

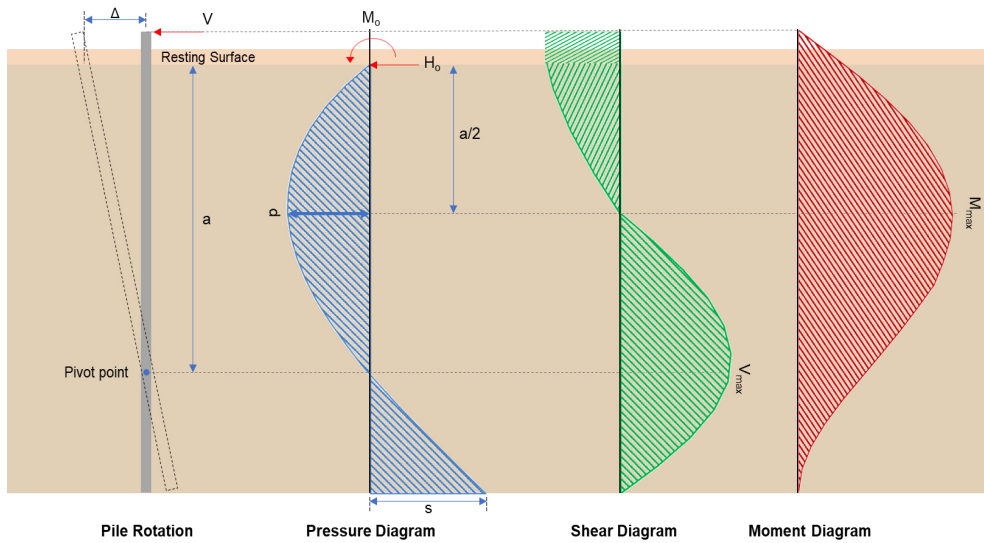
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00066694 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.00093605$$

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,

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

$$H_o = \frac{(-0.885 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14092 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.18 \text{ kipft}) + ((-0.885 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.5764 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.5764 \text{ kipft/ft})}{(-0.14092 \text{ kip/ft})}$$

$$E = 18.282 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5764 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.5764 \text{ kipft/ft})) + (4 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = \frac{(-0.14092 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.5764 \text{ kipft/ft})) + (4 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2251 \text{ ft}$$

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

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

$$V_{max} = ((-0.14092 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.282 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2251 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.282 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2251 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2167 \text{ kip}$$

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

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

$$M_{max} = ((-0.14092 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(18.282 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.282 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.282 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.7694 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.026 \text{ kipft}) + ((-0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0041401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0041401 \text{ kipft/ft})}{(-0.00079618 \text{ kip/ft})}$$

$$E = 5.2 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0041401 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.00079618 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.0041401 \text{ kipft/ft})) + (4 \times (-0.00079618 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3165 \text{ ft}$$

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

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

$$V_{max} = ((-0.00079618 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.3165 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.3165 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0082713 \text{ kip}$$

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

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

$$M_{max} = ((-0.00079618 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(5.2 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.01828 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.519 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.28 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.28 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.519 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0035583$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.519 \text{ kip} \rightarrow 9519 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9519 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.75 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.75 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.75 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.75 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.92 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.2167 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2167 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.038015$$

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

Considering z-direction:

$V_{max} = 0.0082713 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.0082713 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.000074569$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 9.7694 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(9.7694 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.03914$$

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

Considering z-direction:

$M_{max} = 0.01828 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.01828 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.000073236$$

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

REFERENCES	CALCULATIONS	RESULTS
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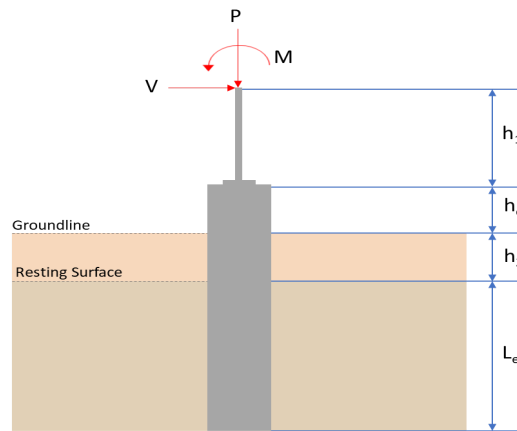
SkyCiv Foundation Design

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

$L = 4.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.401	9.563
V_x (kip)	-0.526	-0.880
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	9.296	16.164

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,

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

$$H_o = \frac{(-0.526 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.083758 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.296 \text{ kipft}) + ((-0.526 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4803 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.5701 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(4.5701 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 4.57 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.57 \text{ ft})}{(4.75 \text{ ft})}$$

$$\text{Ratio} = 0.96211$$

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

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

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

$$q = \frac{(6.401 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40006 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40006 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20003$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.083758 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4803 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4803 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.083758 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.4803 \text{ kipft/ft})) + (4 \times (-0.083758 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2268 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4803 \text{ kipft/ft})) + (3 \times (-0.083758 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.4803 \text{ kipft/ft})) + (2 \times (-0.083758 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20381 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4803 \text{ kipft/ft})) + ((-0.083758 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.68148 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2268 \text{ ft})}{2}$$

$$p_a = 0.24201 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20381 \text{ kip/ft}^2)}{(0.24201 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84215$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

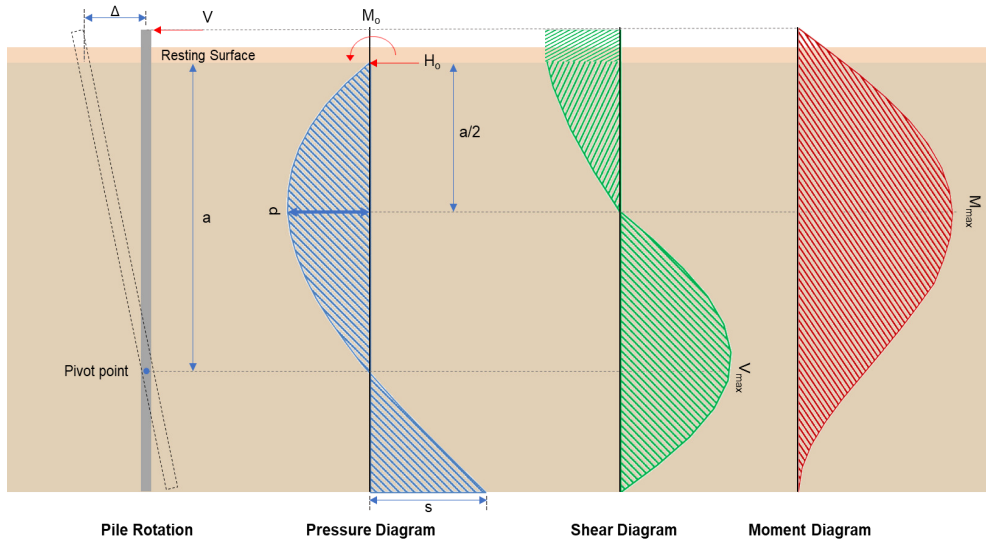
$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.68148 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.88 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.164 \text{ kipft}) + ((-0.88 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.5739 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.5739 \text{ kipft/ft})}{(-0.14013 \text{ kip/ft})}$$

$$E = 18.368 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5739 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.14013 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.5739 \text{ kipft/ft})) + (4 \times (-0.14013 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2249 \text{ ft}$$

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

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

$$V_{max} = ((-0.14013 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.368 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2249 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.368 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2249 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2108 \text{ kip}$$

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

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

$$M_{max} = ((-0.14013 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(18.368 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2249 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.368 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2249 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (18.368 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2249 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.7568 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

Table 22.4.2.1

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

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{9.563 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.278 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.278 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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**

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(9.563 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0035747$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.563 \text{ kip} \rightarrow 9563 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9563 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.76 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.76 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.76 \text{ kip}$$

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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.76 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

$V_{max} = 4.2108 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(4.2108 \text{ kip})}{(110.93 \text{ kip})}$$

$$\text{Ratio} = 0.037961$$

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

Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus

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

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

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

 $\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

 M_n shall be the lesser of: $\phi M_{n,1}$

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

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

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

14.5.2.1b

 $\phi M_{n,2}$

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

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

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

Therefore,

 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction: $M_{max} = 9.7568 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.7568 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.03909$$

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

REFERENCES	CALCULATIONS	RESULTS
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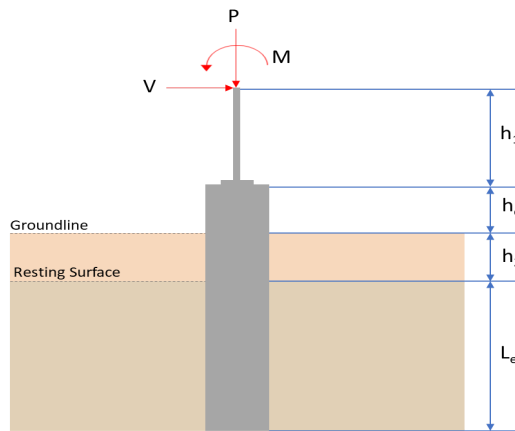
SkyCiv Foundation Design

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

$L = 4.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.374	9.519
V_x (kip)	-0.527	-0.885
V_z (kip)	0.003	0.005
M_x (kipft)	0.015	0.026
M_z (kipft)	9.297	16.181

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,

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

$$H_o = \frac{(-0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.083917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.297 \text{ kipft}) + ((-0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4804 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.5697 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.003 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.015 \text{ kipft}) + ((0.003 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0023885 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.59276 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.5697 \text{ ft}), (0.59276 \text{ ft})]$$

$$L_{e,req} = 4.57 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.75 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.57 \text{ ft})}{(4.75 \text{ ft})}$$

$$Ratio = 0.96211$$

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

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

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

$$q = \frac{(6.374 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39838 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39838 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1992$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.083917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4804 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4804 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.4804 \text{ kipft/ft})) + (4 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2269 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4804 \text{ kipft/ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.4804 \text{ kipft/ft})) + (2 \times (-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20373 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4804 \text{ kipft/ft})) + ((-0.083917 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.68137 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2269 \text{ ft})}{2}$$

$$p_a = 0.24202 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20373 \text{ kip/ft}^2)}{(0.24202 \text{ kip/ft}^2)}$$

$$Ratio = 0.84179$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.68137 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.9563$$

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

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

Considering z-direction:

$H_o = 0.00047771 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0023885 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0023885 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.0023885 \text{ kipft/ft})) + (4 \times (0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3202 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0023885 \text{ kipft/ft})) + (3 \times (0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (0.0023885 \text{ kipft/ft})) + (2 \times (0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.00076031 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0023885 \text{ kipft/ft})) + ((0.00047771 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.0018738 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3202 \text{ ft})}{2}$$

$$p_a = 0.24901 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00076031 \text{ kip/ft}^2)}{(0.24901 \text{ kip/ft}^2)}$$

$$Ratio = 0.0030533$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

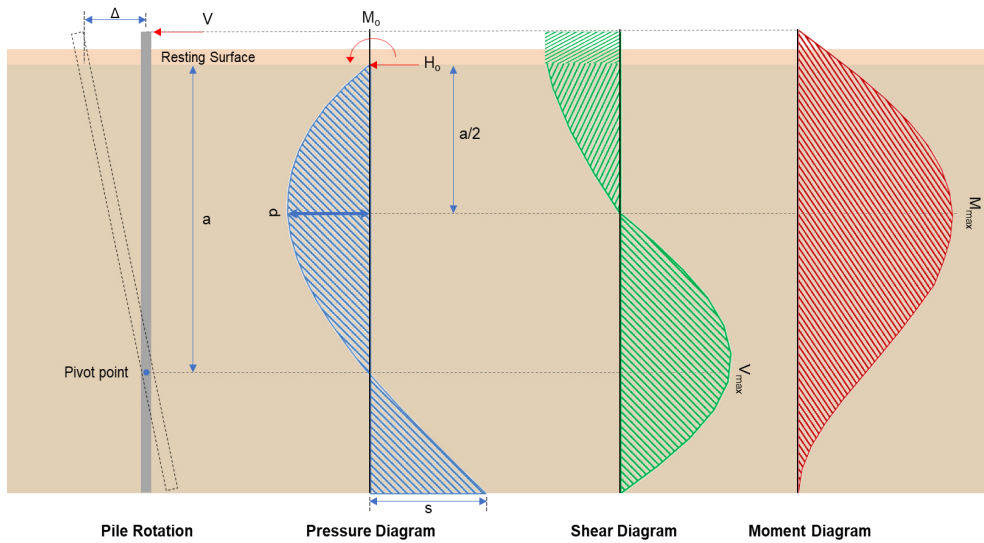
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0018738 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0026299$$

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,

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

$$H_o = \frac{(-0.885 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14092 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.181 \text{ kipft}) + ((-0.885 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.5766 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.5766 \text{ kipft/ft})}{(-0.14092 \text{ kip/ft})}$$

$$E = 18.284 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5766 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.5766 \text{ kipft/ft})) + (4 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = \frac{(-0.14092 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.5766 \text{ kipft/ft})) + (4 \times (-0.14092 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2251 \text{ ft}$$

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

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

$$V_{max} = ((-0.14092 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.284 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2251 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.284 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2251 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2169 \text{ kip}$$

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

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

$$M_{max} = ((-0.14092 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(18.284 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.284 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.284 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2251 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.77 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.026 \text{ kipft}) + ((0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0041401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0041401 \text{ kipft/ft})}{(0.00079618 \text{ kip/ft})}$$

$$E = 5.2 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0041401 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.00079618 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.0041401 \text{ kipft/ft})) + (4 \times (0.00079618 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3165 \text{ ft}$$

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

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

$$V_{max} = ((0.00079618 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.3165 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.3165 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0082713 \text{ kip}$$

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

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

$$M_{max} = ((0.00079618 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(5.2 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.3165 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.01828 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.519 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.28 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.28 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.519 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0035583$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.519 \text{ kip} \rightarrow 9519 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9519 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.75 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.75 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.75 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.75 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.92 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.2169 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2169 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.038017$$

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

Considering z-direction:

$V_{max} = 0.0082713 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.0082713 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.000074569$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 9.77 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(9.77 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.039142$$

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

Considering z-direction:

$M_{max} = 0.01828 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.01828 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.000073236$$

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

REFERENCES	CALCULATIONS	RESULTS
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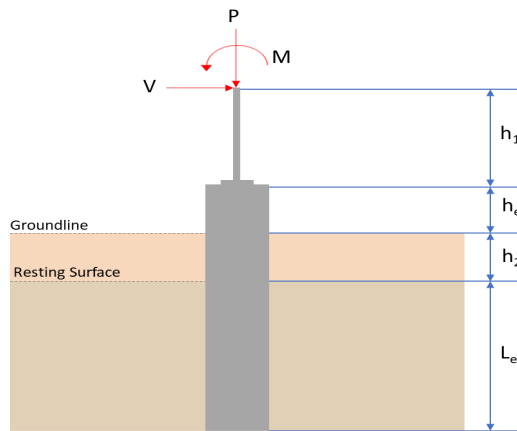
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.445	9.636
V_x (kip)	-0.545	-0.917
V_z (kip)	-0.073	-0.118
M_x (kipft)	-0.421	-0.682
M_z (kipft)	9.748	17.017

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,

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

$$H_o = \frac{(-0.545 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.086783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.748 \text{ kipft}) + ((-0.545 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5522 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.6417 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.073 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.011624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.421 \text{ kipft}) + ((-0.073 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.067038 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6179 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.6417 \text{ ft}), (1.6179 \text{ ft})]$$

$$L_{e,req} = 4.642 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(4.642 \text{ ft})}{(5 \text{ ft})}$$

$$Ratio = 0.9284$$

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

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

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

$$q = \frac{(6.445 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40281 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20141$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.086783 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.5522 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5522 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.5522 \text{ kipft/ft})) + (4 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3988 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5522 \text{ kipft/ft})) + (3 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.5522 \text{ kipft/ft})) + (2 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.19067 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.5522 \text{ kipft/ft})) + ((-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.64093 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3988 \text{ ft})}{2}$$

$$p_a = 0.25491 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19067 \text{ kip/ft}^2)}{(0.25491 \text{ kip/ft}^2)}$$

$$Ratio = 0.74798$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.64093 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.85457$$

Status: **PASS**
Ratio: **0.750**

Status: **PASS**
Ratio: **0.850**

Considering z-direction:

$H_o = -0.011624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.067038 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.067038 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.011624 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.067038 \text{ kipft/ft})) + (4 \times (-0.011624 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.486 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.067038 \text{ kipft/ft})) + (3 \times (-0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 [(3 \times (0.067038 \text{ kipft/ft})) + (2 \times (-0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.0031093 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.067038 \text{ kipft/ft})) + ((-0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.018229 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.486 \text{ ft})}{2}$$

$$p_a = 0.26145 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0031093 \text{ kip/ft}^2)}{(0.26145 \text{ kip/ft}^2)}$$

$$Ratio = 0.011893$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

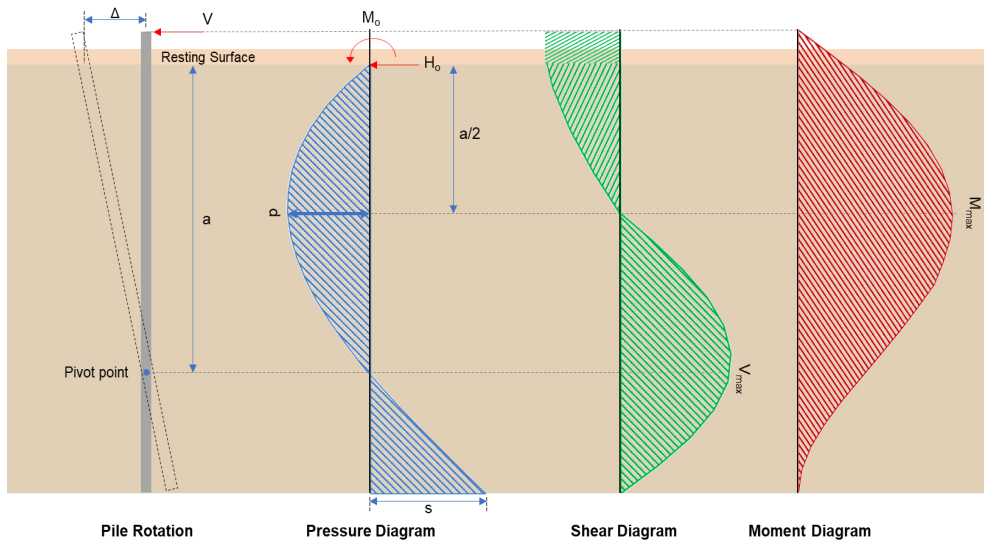
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.018229 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.024306$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.917 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14602 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.017 \text{ kipft}) + ((-0.917 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7097 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.7097 \text{ kipft/ft})}{(-0.14602 \text{ kip/ft})}$$

$$E = 18.557 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7097 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.7097 \text{ kipft/ft})) + (4 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(-0.14602 \text{ kip/ft}) \times (5 \text{ ft})}{(6 \times (2.7097 \text{ kip/ft})) + (4 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3968 \text{ ft}$$

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

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

$$V_{max} = ((-0.14602 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3968 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3968 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2265 \text{ kip}$$

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

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

$$M_{max} = ((-0.14602 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(18.557 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.299 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.682 \text{ kipft}) + ((-0.118 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1086 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1086 \text{ kipft/ft})}{(-0.01879 \text{ kip/ft})}$$

$$E = 5.7797 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1086 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.01879 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.1086 \text{ kipft/ft})) + (4 \times (-0.01879 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4857 \text{ ft}$$

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

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

$$V_{max} = ((-0.01879 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4857 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4857 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20332 \text{ kip}$$

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

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

$$M_{max} = ((-0.01879 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.7797 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.47438 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.636 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.636 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.003602$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.636 \text{ kip} \rightarrow 9636 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9636 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

22.5.5.1.2 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:

$V_{max} = 4.2265 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2265 \text{ kip})}{(110.93 \text{ kip})}$$

$$Ratio = 0.0381$$

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

Considering z-direction:

$V_{max} = 0.20332 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.20332 \text{ kip})}{(110.93 \text{ kip})}$$

$$Ratio = 0.0018329$$

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:

$M_{max} = 10.299 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.299 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.041262$$

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

Considering z-direction:

$M_{max} = 0.47438 \text{ kipft}$ - Maximum moment in the z-direction,

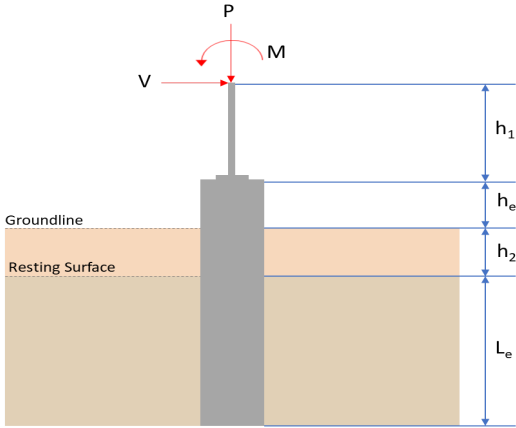
Ratio - Capacity

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

$$Ratio = \frac{(0.47438 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0019006$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1086 1225 1187"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.445</td> <td>9.636</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.545</td> <td>-0.917</td> </tr> <tr> <td>V_z (kip)</td> <td>0.073</td> <td>0.118</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.421</td> <td>0.682</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.748</td> <td>17.017</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength.</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.445	9.636	V_x (kip)	-0.545	-0.917	V_z (kip)	0.073	0.118	M_x (kipft)	0.421	0.682	M_z (kipft)	9.748	17.017	
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																									
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V_z (kip)	0.073	0.118																										
M_x (kipft)	0.421	0.682																										
M_z (kipft)	9.748	17.017																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.545 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.086783 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.748 \text{ kipft}) + ((-0.545 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5522 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.6417 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.073 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.011624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.421 \text{ kipft}) + ((0.073 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.067038 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8829 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.6417 \text{ ft}), (1.8829 \text{ ft})]$$

$$L_{e,req} = 4.642 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.642 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9284$$

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

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

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

$$q = \frac{(6.445 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40281 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20141$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.086783 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.5522 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5522 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.5522 \text{ kipft/ft})) + (4 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3988 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5522 \text{ kipft/ft})) + (3 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.5522 \text{ kipft/ft})) + (2 \times (-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.19067 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.5522 \text{ kipft/ft})) + ((-0.086783 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.64093 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3988 \text{ ft})}{2}$$

$$p_a = 0.25491 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19067 \text{ kip/ft}^2)}{(0.25491 \text{ kip/ft}^2)}$$

$$Ratio = 0.74798$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.64093 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.85457$$

Status: **PASS**
Ratio: **0.750**

Status: **PASS**
Ratio: **0.850**

Considering z-direction:

$H_o = 0.011624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.067038 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.067038 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.011624 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.067038 \text{ kipft/ft})) + (4 \times (0.011624 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.486 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.067038 \text{ kipft/ft})) + (3 \times (0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.067038 \text{ kipft/ft})) + (2 \times (0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.018511 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.067038 \text{ kipft/ft})) + ((0.011624 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.046127 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.486 \text{ ft})}{2}$$

$$p_a = 0.26145 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.018511 \text{ kip/ft}^2)}{(0.26145 \text{ kip/ft}^2)}$$

$$Ratio = 0.070803$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

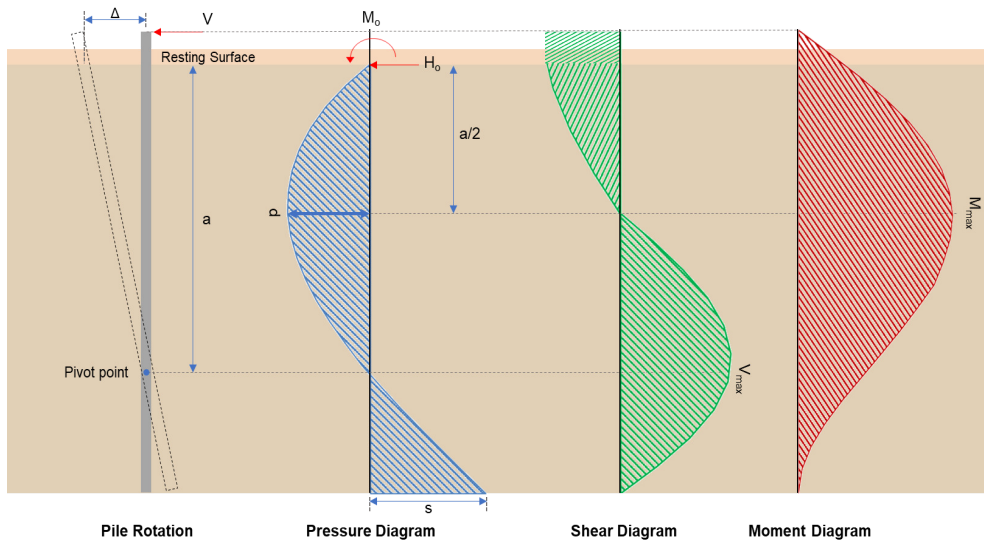
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.046127 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.061503$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.917 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14602 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.017 \text{ kipft}) + ((-0.917 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7097 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.7097 \text{ kipft/ft})}{(-0.14602 \text{ kip/ft})}$$

$$E = 18.557 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7097 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.7097 \text{ kipft/ft})) + (4 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(-0.14602 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.7097 \text{ kipft/ft})) + (4 \times (-0.14602 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3968 \text{ ft}$$

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

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

$$V_{max} = ((-0.14602 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3968 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3968 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2265 \text{ kip}$$

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

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

$$M_{max} = ((-0.14602 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(18.557 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (18.557 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3968 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.299 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.682 \text{ kipft}) + ((0.118 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1086 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1086 \text{ kipft/ft})}{(0.01879 \text{ kip/ft})}$$

$$E = 5.7797 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1086 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.01879 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.1086 \text{ kipft/ft})) + (4 \times (0.01879 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4857 \text{ ft}$$

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

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

$$V_{max} = ((0.01879 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4857 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4857 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20332 \text{ kip}$$

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

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

$$M_{max} = ((0.01879 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.7797 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.7797 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4857 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.47438 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.636 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

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

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

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

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

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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.636 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.003602$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.636 \text{ kip} \rightarrow 9636 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9636 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

22.5.5.1.2 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:

$V_{max} = 4.2265 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.2265 \text{ kip})}{(110.93 \text{ kip})}$$

$$Ratio = 0.0381$$

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

Considering z-direction:

$V_{max} = 0.20332 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.20332 \text{ kip})}{(110.93 \text{ kip})}$$

$$Ratio = 0.0018329$$

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:

$M_{max} = 10.299 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.299 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.041262$$

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

Considering z-direction:

$M_{max} = 0.47438 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47438 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0019006$$

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